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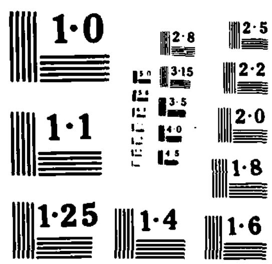
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US Army Corps of Engineers
Los Angeles District

Gila River Basin

Phoenix, Arizona, and Vicinity (Including New River)

New River Dam Foundation Report

October 1985

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20. ABSTRACT (*Continue on reverse side if necessary and identify by block number*)

This report provides a complete and accurate record of foundation conditions underlying New River Dam and its appurtenances. The report also summarizes construction methods employed in excavating, preparing and treating the foundations; describes testing and explorations performed during construction and design modifications required by unforeseen site conditions; and discusses conditions which may require post-construction observation or treatment. Includes 64 plates.

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GILA RIVER BASIN
PHOENIX, ARIZONA AND VICINITY (INCLUDING NEW RIVER)

NEW RIVER DAM
FOUNDATION REPORT

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT
OCTOBER 1985

NEW RIVER DAM FOUNDATION REPORT

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NEW RIVER DAM
PERTINENT DATA

Feature Description	Unit	Data
Drainage area	sq. mi	164
Type of dam	--	compacted earthfill
Main embankment:		
Crest elevation	ft, NGVD	1486.7
Maximum height above streambed	ft	104
Crest length	ft	2,320
Freeboard	ft	5.6
Dike No. 1:		
Crest elevation	ft, NGVD	1486.3
Crest length	ft	7,464
Freeboard	ft	5.2
Maximum height	ft	36
Dike No. 2:		
Crest elevation	ft, NGVD	1484.0
Crest length	ft	256
Freeboard	ft	2.9
Maximum height	ft	9
Spillway:		
Crest elevation	ft, NGVD	1456.2
Crest width	ft	75
Max. water surface elevation	ft, NGVD	1481.1
Max. spillway outflow	ft^3/s	29,850
Outlet conduit:		
Interior dimension	ft	9.5 H x 6.25 W
Length	ft	433
Inlet elevation	ft, NGVD	1389.25
Outlet elevation	ft, NGVD	1386.31
Max. outlet outflow	ft^3/s	3,150
Energy dissipator:		
Length	ft	60.98
Width	ft	31.0
Floor elevation	ft, NGVD	1372.0
Wall height	ft	22.0
Outlet channel:		
Base width	ft	16.0
Sideslope	--	2.5 H to 1V
Levee height	ft	8.0 - 1.0
Length	ft	730.32

NEW RIVER DAM
PERTINENT DATA (Continued)

Feature Description	Unit	Data
Reservoir area:		
Spillway crest	acres	1,780
Max. water surface	acres	2,900
Capacity (gross):		
Spillway crest	ac ft	43,520
Max. water surface	ac ft	102,520
Storage allocation below spillway crest:		
Flood control (net)	ac ft	38,600
Sedimentation	ac ft	4,920
Standard project flood:		
Total volume	ac ft	49,300
Peak inflow	ft^3/s	45,000
Peak outflow	ft^3/s	2,665
Drawdown time (to empty)	days	10.1
Probable maximum flood:		
Total volume	ac ft	105,000
Peak inflow	ft^3/s	144,000
Peak outflow	ft^3/s	33,000
Drawdown time (to spillway crest)	days	3.3

NEW RIVER DAM
FOUNDATION REPORT

1. Introduction

Purpose and Scope

1.01 The New River Dam Foundation report was prepared by Robert L. Thurman, project geologist, to satisfy the requirements set forth in ER 1110-1-1801 dated 15 December 1981. The report provides a complete and accurate record of the foundation conditions underlying the dam and its appurtenances. The report also summarizes construction methods employed in excavating, preparing and treating the foundations; describes testing and explorations performed during construction and design modifications required by unforseen site conditions; and discusses conditions which may require post-construction observation or treatment. Pre-construction investigations and analyses are briefly presented as they are detailed in the Phase II, General Design Memorandum (GDM), Appendix 1, dated November 1982. Pertinent construction drawings are presented with geotechnically related changes and deviations shown in red. The official "As-Built" drawings were not available at the time of publication. Additional drawings describe the foundation geology of the principal structures. Photographs of significant features are also presented. Information on the design and construction of the dam and dike embankments is presented in the New River Dam Embankment Performance and Criteria Report (U.S. Army Corps of Engineers, in prep.).

1.02 The information presented in the foundation report is intended for use: (1) in planning additional foundation treatment should the need arise after project completion; (2) in evaluating the cause of distress or partial failure of a structure, and in planning remedial action should such a situation occur as a result of foundation deficiencies; (3) for guidance in planning foundation explorations, and in anticipating foundation problems for future comparable construction projects; (4) as an information base in determining the validity of potential claims by the Contractor in connection with foundation conditions; and (5) as a part of the permanent collection of project engineering data.

Project Location and Description

1.03 The project is located in Maricopa County, Arizona, approximately 22 miles northwest of downtown Phoenix and about 6 miles west of Interstate 17 (Black Canyon Highway), see plate 1. The dam is located on the New River about 4-1/2 miles south of Carefree Highway and spans a relatively narrow valley between the East and West Wing Mountains. The dam controls a drainage area of approximately 164 square miles and is part of the Phoenix, Arizona and Vicinity (including New River) Flood Control Project.

1.04 The New River Dam project consists of the following features:

a. A zoned earthfill dam approximately 104 feet high (maximum) and approximately 2320 feet long at the crest. The crest elevation is 1486.7 feet (without settlement allowances) above National Geodetic Vertical Datum (NGVD).

b. A zoned earthfill dike, approximately 36 feet high (maximum) and approximately 7464 feet long at the crest, about 1.7 miles northwest of the right abutment of the dam. The crest elevation is 1486.0 feet NGVD (without settlement allowances).

c. Another earthfill dike, approximately 9 feet high and approximately 256 feet long at the crest, about 1/2 mile northeast of the left abutment of the dam. The crest elevation is 1484.0 feet NGVD (without settlement allowances).

d. An ungated outlet, 6.25 feet wide by 9.5 feet high, near the base of the left abutment.

e. A detached unlined spillway, 75 feet wide at the base, about 700 feet northwest of the right abutment of the dam. A general plan of the project is shown on plate 1. Views of the completed project are shown in photographs 1 through 15.

Project History and Authorization

1.05 The Phoenix, Arizona and Vicinity (including New River) Flood Control District was authorized by the Flood Control Act of 1965 (Public Law No. 89-298, 89th Congress) to help alleviate the flood hazard that exists in the Phoenix metropolitan area. Postauthorization studies were initiated in the spring of 1969 and during Phase I studies, a combination structural-nonstructural plan was determined to be the best solution to the flood problem in the area. The approved plan, which differs from the authorized plan, involves the construction of four earthfill dams (Dreamy Draw Dam, completed in 1973; Cave Buttes Dam, completed in 1980; Adobe Dam, completed in 1982; and New River Dam, completed in 1985) and the construction of the Arizona Canal diversion channel.

1.06 Two alternate damsites were geotechnically investigated and evaluated for the New River project. The first site considered was explored between March 1970 and January 1972 and discussed in the Phase I GDM dated March 1976. This site, originally designated the interim report site, was renamed the Phase I site in the Phase II GDM. The second and eventual construction site, originally designated alternative site no. 1 but subsequently referred to as the Phase II site, was located about 1500 feet downstream from the Phase I site. This site was explored in detail for the Phase I GDM between November 1979 and May 1981. Pre-construction explorations done to augment the design studies were made in June 1982. This additional work, although not included in the GDM, was included in the contract plans. Geotechnical considerations had no significant influence on site selection. The rationale for selecting the Phase II site was made on the basis of economics. All explorations pertinent to the foundation and excavation studies, including those made in 1984 during construction, are presented in this report.

1.07 The New River Dam construction contract was advertised under bid reference No. DACW09-83-B-0016. Bids were opened on 2 August 1983 and M. M. Sundt Construction Company of Tucson, Arizona was awarded Contract No. DACW09-

83-C-0050 on August 1983 with a total bid of $10,250,000.00. The remaining bids ranged from $10,340,000.00 to $18,497,400.00. The government estimate was $12,331,105.00. Construction began in October 1983 and the dam was officially dedicated on 8 February 1985, approximately 6 months in advance of the required completion date. The final contract cost was $11,749,344.60; the increase due primarily to additional processing procedures required to obtain adequate and acceptable Type I and Type II stone protection. Table 1 provides a complete list of the final contract bid items and table 2 provides a list of geotechnially related contract modifications and costs. Descriptions of these modifications can be found under the appropriate sections of the text.

Key Resident and Design Staff

1.08 The following is a partial list of Corps of Engineers personnel responsible for the design and construction of New River Dam:

Engineering Division:	
Project Manager	Stan Lutz
Civil Design	Yan Bahaudin
Technical Specialist	Albert Honda
Soils Design	Rudy Roodsari
Soils Design	Ted Ingersoll
Geology	Vern Minor
Geology	Bob Thurman
Materials and Investigation	Bill Halczak
Hydraulics	Edward Chew
Environmental Planning	Lynn Alm[illegible]
Cultural Resources	Helen Wells
Landscape Architecture	Mike Evasovic
Survey	Bud Anderson
Construction Division:	
Resident Engineer	Neil Erwin
Project Engineer	Cpt. Bob Dunne
Office Engineer	Mike Ternak
Office Technician	Satsuki Carrington
Assistant Project Engineer/ Field Superintendent	Joe Salinez
Laboratory Chief	Rick Flott

Contract Supervision and Quality Control

1.09 In addition to the Construction Division personnel listed in paragraph 1.08, the New River Dam project office maintained a permanent staff of two field inspectors and three laboratory technicians, supplemented by additional Los Angeles District, other District or military personnel when required. Quality control over embankment placement was the responsibility of the Corps of Engineers. The Contractor had quality control over the other aspects of construction, which were nevertheless under constant inspection by the Project Office staff. Engineering Division Geotechnical Branch personnel were

actively involved in the supervision and inspection of geotechnically related items such as excavation for the dam embankment and appurtenances, foundation preparation and treatment and initial material placement. Supervision and inspection of the foundation drilling and grouting program was the responsibility of Geotechnical Branch personnel.

Subcontractors

1.10 The prime contractor (M. M. Sundt) subcontracted several aspects related to the construction of New River Dam. The major subcontractors performing geotechnically related tasks were as follows:

a. Brooks Hersey, Tempe, Arizona, the project surveyors.

b. W. G. Jaques Co., Des Moines, Iowa, the drilling and grouting subcontractor.

c. Western Technologies, Inc., Phoenix, Arizona, performed materials testings.

d. United Metro Division, Phoenix, Arizona, the concrete suppliers for the project.

2. Geology

Regional Topography

2.01 The New River Dam project is located in that portion of Arizona referred to by Menges (1983) as the Gila Lowland Section of the Sonoran Desert Subprovince, Southern Basin and Range Physiographic Province. The Province is characterized by broad, gently sloping connected valleys or plains bounded by moderately high, rugged mountain ranges rising abruptly to maximum heights of several thousand feet above the fairly flat valley floors. The project is at the southern edge of a topographic and structural basin and is bounded to the southeast and southwest by the low-lying East and West Wing Mountains, respectively, and to the east by an unnamed group of peaks. The broad alluvial plain to the north and west of the project extends up to the New River and Hieroglyphic Mountains. The dam embankment spans a relatively narrow valley between the East and West Wing Mountains at the northern edge of Deer Valley, a small undissected tributary valley within the larger alluvial plain of the Salt River Valley.

Regional Geology

2.02 The rock types found in the mountainous areas that border the project consist of: (1) an igneous and metamorphic basement complex composed predominantly of Precambrian granite and related crystalline rocks with lesser amounts of schist and gneiss; (2) Cretaceous to Tertiary intrusive igneous rocks, consisting mainly of granite and monzonite; and (3) Tertiary volcanic rocks in the form of basalt and andesite with local accumulations of tuff, flow breccia and agglomerate. The basement complex is extensively exposed along the eastern and southeastern margins of the project. Elsewhere, particularly along the southwestern margin of the project, Tertiary age lava flows rest unconformably upon the basement complex. Exposures of intrusive igneous rocks are limited, occurring mainly in the mountains to the east.

2.03 Older sediments that constitute the valley fill are Quaternary in age and are composed mainly of poorly- to well-consolidated gravel, sand, silt and clay, representing several environments of deposition. The constituent materials were eroded from the adjacent mountain masses by stream and sheet runoff. Calcium carbonate cementation is common and considerable caliche is present near the mountain fronts. Recent (Quaternary) alluvium, consisting mainly of unconsolidated sand and gravel, fills the channels of the main stream courses and the tributaries associated with flood plain washes. The total thickness of the alluvial materials varies from zero along the mountain fronts to depths exceeding 1200 feet under the valley interior (Cooley, 1973).

Geologic History

2.04 The Cenozoic history of southwestern Arizona was ushered in during the Laramide orogeny, which began in the late Cretaceous, some 90 million years ago, and continued for about 40 million years, into the early Tertiary. This portion of the State was severely affected by this tectonic episode which was

characterized by regional uplift, eruption of rhyolitic to andesitic volcanic rocks, and by intrusion of several large bodies or plutons of igneous rock of a granitic composition. Following the cessation of the Laramide orogeny, southwestern Arizona was an area of general magmatic quiescence. During this time, a broad erosional surface dipping northeastward was developed and deposition of sediments in localized interior drainage basins occurred.

2.05 Another period of widespread tectonism began approximately 30 million years ago during the late Oligocene and lasted about 10 million years, into the middle Miocene. This episode, referred to as the "mid-Tertiary orogeny" (Scarborough, 1979), was accompanied by extrusion of great quantities of rhyolitic to andesitic tuffs, breccias and flows, and deposition of thick sequences of clastic sediments in newly formed interior drainage basins. Rocks deposited during and preceding this event were subject to low angle normal faulting, steep tilting and local folding. Many of the older extrusive rocks in the Salt River Valley area were products of this orogeny. With the waning of the mid-Tertiary orogeny, a profound unconformable surface was developed. Topographic lows became sites for fanglomerate and lacustrine deposition. Tuff beds and extrusive flows intercalated in these sedimentary deposits indicate continued though minor volcanic activity.

2.06 During the late Miocene, approximately 15 million years ago, subsidence, high angle normal block-faulting and erosion occurred in southwestern Arizona, which disrupted all earlier landforms. This resulted in the development of a typical basin and range structure of mountain-forming horsts separated by valleys underlain by grabens or half-grabens. Deposition of sediments began in the basins as the basins were formed. In the Salt River Valley area, these sediments were deposited under oxidizing conditions in fluviatile and lacustrine environments and consisted of clastics and evaporite sequences. Included in the sedimentary sequence are occasional interbeds of extrusive volcanic rocks of basaltic composition. Approximately 10 million years ago, faulting began to wane and sedimentation in previously separate interior basins began to coalesce.

Site Topography

2.07 The New River, an ephemeral stream, flows generally south from its headwaters in the New River Mountains across a broad gently sloping valley to the dam, a distance of approximately 24 miles. The stream gradient in the vicinity of the project is about 10 feet per mile. At the dam, the valley narrows considerably, to a width of about 2000 feet. The dam embankment spans the New River between the West Wing Mountains, which form the right abutment, and Keefer Hill, a westward projection of the East Wing Mountains, which form the left abutment (see pl. 1). The mountains are characteristically steep and rugged although they attain only moderate heights. Elevations in the project area range from 1390 feet in the streambed up to 2000 feet at the crests of the surrounding mountains. South of the dam, the New River flows through Deer Valley for a distance of approximately 8 miles before merging with Skunk Creek, then flows about 8 miles further downstream before merging with the Agua Fria River.

Site Geology

2.08 The geological formations present within the project area consist generally of: (1) Precambrian granitic rocks; (2) Tertiary volcanic rocks; and (3) Quaternary alluvial deposits. This section presents a general discussion of the site geology. Detailed discussions of the geology and foundation conditions are presented with each project feature. More comprehensive petrographic work conducted by the South Pacific Division (SPD) laboratory on 15 selected rock samples collected during construction resulted in an expanded, more detailed, and sometimes revised rock classification scheme (see pl. 28). The rock names used in this report are based primarily on petrographic analyses. However, color and textural characteristics were occasionally considered in the classification process to more clearly distinguish between chemically similar rock types. Rock unit names and/or designations used in the original contract plans (see pl. 2) were changed to conform to the lithologic classifications used in this report. The petrographic analyses of the various rock samples are inclosed as attachments 1 and 2.

GEOLOGIC FORMATIONS

Granitic Rocks

2.09 The Precambrian age granitic rocks, composed primarily of granite, diorite, and related crystalline rocks, are extensively exposed in the East Wing Mountains, including Keefer Hill, and underlie the outlet works and a portion of the dam foundation on the east side of the valley. These rocks are collectively referred to as the Precambrian basement complex in this report. Granite and diorite are the dominant rock types present and appear to be of plutonic origin. The granite is characterized by its medium- to coarse-grained texture, small percentage of mafic minerals and light gray to reddish-brown color. The diorite, found in close association with the granite, is characterized by its medium- to coarse-grained texture, mottled appearance due to a high percentage of mafic minerals, and medium to whitish-gray color. Scattered occurrences of a fine- to medium-grained, mottled, medium to dark gray quartz diorite may represent a possible postmagmatic alteration of material near the margins of the diorite pluton. The granite and diorite have been intruded in numerous locations by dikes of a medium gray to black, aphanitic granitic rock which also frequently appears as inclusions within the surrounding rock mass.

Volcanic Rocks

2.10 Tertiary-age volcanic rocks, composed of andesite, several varieties of tuff, flow breccia and agglomerate are extensively exposed in the West Wing Moutains. The dominant rock type, a light to medium gray aphanitic andesite, is present on the right abutment of the dam and in the spillway excavation and also underlies a portion of the dam foundation on the west side of the valley. A reddish-brown to pinkish-gray porphyritic andesite outcrops in the northwestern part of the West Wing Mountains and underlies the south abutment of dike no. 1. Associated with the aphanitic andesite is a reddish-brown volcanic cinder flow breccia, which is present as a fairly continuous layer

below the andesite on the southeast flank of the mountains near the downstream end of the spillway excavation. The breccia also locally caps the andesite and infills joints in the bedrock on the northeast flank of the mountains immediately upstream of the right abutment and near the upstream end of the spillway excavation. The tuff sequence, composed of a series of pyroclastic rocks reflecting different modes of origin which have undergone varying degrees of consolidation, is exposed in the spillway excavation below the flow breccia unit and forms a prominent ridge with columnar-type jointing along the southeastern flank of the West Wing Mountains. Rock unit lithologies range from well-stratified welded ash-fall tuffs to non-stratified slightly welded ash-flow tuffs. A tuffaceous agglomerate, exposed along the northeast flank of the mountains upstream of the right abutment, was found to locally cap the andesite bedrock underlying the dam foundation on the west side of the valley.

Alluvium

2.11 The Quaternary-age alluvium can generally be designated as either older poorly- to well-consolidated valley fill, alluvial fan and flood plain deposits; or younger unconsolidated stream channel and tributary wash deposits. The older Quaternary alluvium also includes the usually thin spotty veneer of residual soil and slope wash found on the slopes of the East and West Wing Mountains; colluvium, consisting of desert varnished andesite blocks and rubble, which caps the hills in the vicinity of the right abutment and the spillway excavation; and rounded granite boulders which mantle the slopes and crests of the hills north of the left abutment and cover the lower slopes downstream of the left abutment. The valley floor is covered principally by finer-grained flood plain deposits consisting mostly of sands and silts which attain a maximum thickness of approximately 9 feet. The underlying coarser-grained valley fill deposits, consisting mainly of clays, sands and gravels with numerous layers and lenses of older stream channel cobbles and boulders present to a depth of about 25 feet, extend down to bedrock, which is at known maximum depths ranging from 136 to 144 feet beneath the dam foundation near the center of the valley. Erratic, near-surface zones of caliche cementation are common on the east side of the valley above the shallow granitic bedrock pediment. The alluvium covering the slopes of the mountains is generally less than 2 feet thick.

Faulting and Seismicity

2.12 The greatest concentration of faults, particularly Quaternary faults, in the State of Arizona occur in a poorly-defined band stretching diagonally from northwest to southeast across the state, generally coinciding with areas of historical seismicity (Menges, 1983). Most faults generally exhibit steep dips and normal separation. Quaternary faults are rare in southwestern Arizona and none have been identified in the vicinity of the New River Dam project (Pearthree and others, 1983).

2.13 The closest fault system to the project is the 45-mile-long Verde fault system, located approximately 45 miles to the northeast at its southerly extent. This system consits of several splays and segments, the longest and most continuous of which is the 17-mile-long central segment. A maximum

credible earthquake of Richter magnitude 7.0 could be produced by movement over the total length of the Verde fault system, resulting in a maximum bedrock acceleration of approximately 0.08 g at the dam (U.S. Army Corps of Engineers, 1979). However, the largest earthquake ever recorded to date near the Verde system was the 1976 Chino Valley event with a Richter magnitude of 5.1 and an epicenter location about 65 miles north of the project. This would have produced a ground acceleration of less than 0.01 g at the dam (U.S. Army Corps of Engineers, 1979). This fault system has shown evidence of Quaternary movement but no historic or Holocene surface ruptures have been recorded.

2.14 The Basin and Range province in southwestern Arizona has been considered to be tectonically inactive due in part to the low levels of historical seismicity and the extensive pedimentation of mountain blocks (Pearthree and others, 1983). Evidence of average regional recurrence intervals between surface-rupturing earthquakes over the last 15,000 years range from 3500 years to possibly 15,000 years or more (Pearthree and others, 1983), indicating a lesser degree of seismic hazard potential for this portion of the state. New River Dam is located in Zone 1 of the Seismic Zone Map of the Contiguous States (U.S. Army Corps of Engineers, 1983), an area of low seismicity. Only five earthquakes with maximum epicentral intensities between V and VI on the Modified Mercalli intensity scale have been reported within a 50-mile radius of the project since 1871 (Dubois and others, 1982). Research published by the United States Geological Survey indicates that the project has a 90-percent probability of experiencing bedrock accelerations no greater than 0.04 g in 50 years (Algermissen and Perkins, 1976). A bedrock acceleration of 0.08 g produced by a magnitude 7.0 earthquake on the Verde fault system would require simultaneous movement on all segments and is not likely to occur during the design life of the project.

Ground-Water

2.15 Ground water information for the New River Dam project was obtained from three sources: (1) water well records obtained from the State of Arizona Department of Water Resources, (2) subsurface investigations conducted prior to and during construction, and (3) observation wells installed after construction of the dam embankment had been completed. Using the most recent water level data available, a ground water basin profile through the project area was developed and is shown in figure 1. Within this 5 mile reach, the water table gradient changes considerably due to varying subsurface conditions and the intensity of ground water development.

2.16 In the basin area upstream of the dam, ground water withdrawal has been minimal and the water table gradient is flatter than the corresponding topographic slope, based on water level data provided by the following 3 wells. In 1985, ground water was measured at a depth of 100 feet (elevation, 1320 feet) in well (A-5-1) 25bca, approximately 1 mile upstream, and at a depth of 121 feet (elevation, 1342 feet) in observation well no. 3, about 2-1/4 miles upstream. In 1984, ground water was measured at a depth of 212 feet (elevation, 1362 feet) in well (A-5-1) 10aab, approximately 4-1/2 miles upstream of the dam and about 1-1/2 miles north of dike no. 1.

2.17 At the dam, the ground water table appears to rise significantly to a known maximum elevation of 1357 feet, based on information from observation wells 1 and 2. In 1985, ground water was measured at a depth of 36 feet (elevation, 1351 feet) in observation well no. 1, 350 feet downstream of dam station 24+50, and at a depth of 43 feet (elevation, 1357 feet) in observation well no. 2, 305 feet upstream of dam station 24+50. However, since both wells are fully perforated below a depth of 25 feet, these water levels may be influenced to some extent by any localized zones of perched or semiperched ground water. In fact, underflow from nearby ephemeral borrow ponds which "cascaded" briefly into well no. 2, indicates the presence of at least one separate zone of saturated alluvium. Measurements of water levels and hole depths in observation wells 1 and 2 in September 1985 indicate that these wells may be plugged and that the "static" water levels may not reflect true ground water conditions. Beneath the dam, preconstruction subsurface borings encountered ground water between elevations of approximately 1300 and 1340 feet. These variations may be due to seasonal fluctuations, perched conditions or a possible ground water mound condition beneath the active stream channel. In any event, the overall higher ground water table at the dam is probably caused by shallow bedrock constrictions in the relatively narrow confines of the valley.

2.18 South of the project, in Deer Valley, the water table declines rapidly to depths exceeding 300 feet due to intense ground water development for agricultural as well as residential and industrial purposes. In 1984, ground water was measured at a depth of 351 feet (approximate elevation, 1029 feet) in well (A-4-1) 1baa, approximately one mile south of the dam.

Subsidence

2.19 Surface subsidence and associated earth fissure development have occurred in the Phoenix metropolitan area as a result of major ground water declines (Laney, 1975). Long-term survey data are not available to determine if subsidence has occurred at the project. However, subsidence has probably been negligible due to the relatively shallow depth to bedrock and the lack of any extensive ground water development and should not pose any future problems for the dam embankment and appurtenances. The closest occurrences of measured subsidence has been in Deer Valley along portions of Beardsley Road west of Interstate 17 (Winikka, 1984). The maximum amount of subsidence detected between 1967 and 1981 has been 0.45 foot at 83rd Avenue, approximately 4-1/2 miles south of the dam. Subsidence would be expected in this area where the alluvial materials are much thicker and where ground water declines of up to 300 feet have occurred in the past 30 to 40 years.

2.20 Earth fissures have not been observed in the project area or in Deer Valley. The closest occurrences are about 15 miles to the southwest in the vicinity of Luke Air Force Base, where 1 to 3 feet of subsidence has been detected or estimated (Schumann, 1974).

3. Foundation Exploration

Investigations Prior to Construction

3.01 Subsurface geotechnical investigations were conducted at the New River damsite for the foundation of the dam, dikes, outlet works, three alternative spillways, and potential borrow areas to determine design and cost data. The geotechnical investigations consisted of geologic mapping, bucket-type power auger drilling, diamond core drilling, dozer and backhoe trenching, shallow seismic refraction surveys, and field and laboratory testing. The pre-construction investigations were initiated in 1971 but only a limited amount of field work, consisting of geologic reconnaissance and preliminary mapping plus one bucket auger test hole and one dozer trench, was performed. The balance of the field investigations were conducted between November 1979 and May 1981. Additional work was done in June 1982 to supplement and clarify data on subsurface conditions discussed in the Phase II GDM. This information is presented in the project construction plans and specifications. Plate 2 depicts the general site geology within the project area. The geotechnical investigations performed are summarized briefly in the following paragraphs. Additional information, including the borrow area exploration data and test results, is available in the Phase II GDM.

DAM FOUNDATION

Main Embankment

3.02 The investigation of the streambed portion of the dam foundation consisted of geologic mapping, drilling 8 core holes (DD-14 through DD-16, DD-17a, DD-18, DD-20, DD-28 and DD-34), 2 rockbit holes (DD-17 and DD-44), and 7 bucket auger test holes, excavating 26 backhoe or dozer trenches (including TT-10 through TT-14, TT-46 through TT-54, TT82-1 and TT82-6); and conducting 7 seismic refraction surveys (lines 1, 2, and 11 through 15). The locations of the core holes are shown on plate 2, the locations of the test holes are shown on plate 3, the locations of the seismic refraction lines are shown on plate 4, and the locations of the test trenches are shown on plates 3 and 12. The geologic logs of the core holes are summarized on plate 6, the soils logs of the test holes and 10 dozer trenches are shown on plates 7 through 9, time-distance curves and velocity profiles obtained from the interpretation of data from the geophysical explorations are shown on plate 11, and the geologic logs of 16 test trenches are shown on plates 12 through 14. A generalized geologic profile along the centerline of the dam depicting anticipated subsurface conditions prior to construction is shown on plate 5.

3.03 The core holes were drilled to determine the depth, type and quality of the bedrock, and the rock mass permeability. An unsuccessful attempt to reach bedrock was made during drilling of rockbit hole DD-17 while DD-44 was drilled to investigate the type of material encountered at refusal depth in a nearby test hole. In each test hole, representative disturbed samples of foundation material were obtained at 3-foot intervals or at each change in soil type for classification tests. The test trenches were excavated to determine the near-surface foundation conditions by visual examination, permeability testing, mass gradation sampling and in-situ density testing using the sand displacement

or large-scale water displacement method. Dozer trenches TT82-1 and TT82-6 were excavated to more accurately determine the depths to competent foundation rock and to assess the excavation requirements of the foundation materials. The seismic refraction surveys were intended to yield information on subsurface P-wave velocities and the overburden/bedrock contact. Results from survey lines 1, 2, 11 and 13 were inconclusive when compared with subsequent subsurface information obtained by drilling and trenching. It appeared the subsurface velocity profiles may have actually reflected the depth of bedrock weathering and that density contrasts between the more consolidated alluvium and weathered granitic bedrock were too minimal to be recorded.

Left Abutment

3.04 The investigation of the left abutment consisted of geologic mapping, drilling two inclined core holes, DD-19 and DD-21, and excavating two test trenches, TT82-4 and TT82-5, with a D9 dozer. The locations of the core holes are shown on plate 2 and the geologic logs of the core holes are summarized on plate 10. The locations of the test trenches are shown on plate 12 and the geologic logs of the test trenches are shown on plate 13. The core holes were drilled to determine: (1) the subsurface geologic structure, (2) the rock mass permeability, and (3) general abutment excavation requirements. The purpose of the trenching program was to more accurately determine the depth to sound foundation rock and to assess the excavation characteristics of the foundation materials.

Right Abutment

3.05 The investigation of the right abutment consisted of geologic mapping and drilling two inclined core holes, DD-22 and DD-45. The locations of the core holes are shown on plate 2 and the geologic logs of the core holes are summarized on plate 10. The core holes were drilled to determine: (1) the subsurface geologic structure, (2) the rock mass permeability, and (3) general abutment excavation requirements.

OUTLET WORKS

3.06 Both abutments were investigated as possible locations for the outlet works. The right abutment site was investigated by geologic mapping and 14 backhoe trenches. The trenches were excavated to determine the depth of the overburden/bedrock contact and the subsurface geologic structure. The left abutment site was investigated by geologic mapping, 4 core holes (DD-32 through DD-35), 21 backhoe trenches (TT-9 through TT-14, TT-42, TT-44, TT-45, and TT-81 through TT-92), 2 dozer trenches (TT82-2 and TT82-3) and 1 seismic refraction survey (line 12). The core holes and survey line were located along a preliminary alinement near the toe of the abutment while the trenches were excavated along or across the as-built outlet works alinement. The locations of the core holes and test trenches are shown on plate 12 and the location of the seismic refraction line is shown on plate 4. The geologic logs of the test trenches are shown on plates 13 through 16, the geologic logs of the core holes are summarized on plate 17, and the time-distance curve and velocity profile obtained from the interpretation of data from the survey line

is shown on plate 11. A generalized geologic profile along the outlet works centerline depicting anticipated subsurface conditions prior to construction is shown on plate 12.

3.07 The core holes were drilled to determine the depth to bedrock and the competency of the bedrock foundation under the proposed conduit structure. The dozer trenches, TT82-2 and TT82-3, were excavated to determine the competency and excavation characteristics of the bedrock along the as-built conduit alinement. The backhoe trenches were excavated to more accurately develop bedrock profiles along the entire outlet works alinement while seismic refraction survey line 12 was conducted to yield information on bedrock P-wave velocities and possible excavation requirements. The left abutment site was selected because the outlet works conduit and energy dissipator could be founded entirely on bedrock without having to resort to excessive rock removal and the creation of possible slope stability problems.

SPILLWAY

3.08 In addition to exploration at the constructed spillway site (alternate site no. 2), subsurface investigations were conducted at two additional sites. Spillway no. 1, approximately 2400 feet north east of the left abutment, was investigated by geologic mapping, 3 core holes (DD-9 through DD-11) and 3 seismic refraction surveys (lines 6 through 8). Spillway no. 3, on a ridge adjacent to the left abutment of the dam, was investigated by geologic mapping, 9 core holes and 3 seismic refraction surveys. The design site was investigated by geologic mapping, 14 core holes (DD-13, DD-36 through DD-43 and DD-46 through DD-50), 20 backhoe trenches (TT-58 through TT-77) two D9 dozer trenches (TT-93 and TT-94) and two seismic refraction surveys (lines 9 and 10). The locations of the core holes, and locations and geologic logs of the dozer trenches are shown on plate 18. The location and geologic logs of the backhoe trenches are shown on plate 19 and the locations of the seismic refraction lines are shown on plate 4. The geologic logs of the core holes are summarized on plates 21 and 22. Time-distance curves and velocity profiles obtained from the interpretation of data from the geophysical explorations are shown on plate 23. A generalized geologic profile along the spillway centerline and cross sections which depict anticipated subsurface conditions prior to construction are shown on plate 20. Investigations were performed to: (1) determine the type and quality of bedrock present; (2) assess the excavation requirements and disposition of the excavated materials; (3) evaluate the need for a concrete lining or other treatment; and (4) assess the suitability of the site for construction of a spillway. The design site was selected primarily on the basis of a more desirable downstream flow pattern, both in relation to the embankment and outlet works and because of reduced land acquisition requirements.

DIKE NO. 1

3.09 The investigations for dike no. 1 consisted of geologic mapping, drilling 5 bucket auger test holes, excavating 7 backhoe or dozer test trenches, and conducting 8 seismic refraction surveys. The survey lines and 4 test holes were located along a preliminary alinement for the dike. The locations of the test holes, test trenches and seismic refraction lines are

shown on plate 24. The soils logs of the test holes and test trenches are shown on plate 25. Time-distance curves and a velocity profile obtained from the interpretation of data from the geophysical explorations are shown on plate 26. In the test holes and trenches, representative disturbed samples of the foundation materials were obtained at 3-foot intervals or at each change in soil type for classification tests. Samples were also taken of caliche-cemented materials to determine their soluble salts content.

DIKE NO. 2

3.10 The investigations conducted for spillway no. 1 (see paragraph 3.08) were also used to evaluate foundation conditions for dike no. 2. The locations of the core holes are shown on plate 2, the locations of the seismic refraction lines are shown on plate 4 and the geologic logs of the core holes are summarized on plate 27. Time-distance curves and velocity profiles obtained from the interpretation of data from the geophysical explorations are shown on plate 23.

Investigations During Construction

3.11 Investigations made during construction consisted of drilling 4 NW-size exploratory grout (core) holes, 100C through 103C, and 1 NW-size exploratory core hole, 104C, in the core trench excavation. The locations of the exploratory holes are shown in red on plates 29, 31, 32, and 33. The geologic logs of the exploratory holes are shown in figures 3 through 7. Pressure test and grouting data for each exploratory hole are shown in table 8. The exploratory grout holes were drilled to check the effectiveness of the grouting program and to further explore the subsurface geologic structure beneath the dam foundation. Core hole 104C was drilled to explore the subsurface geologic structure of the shear zone and the rock mass permeability.

Investigations After Construction

OBSERVATION WELLS

3.12 Investigations conducted after the dam embankment had been completed consisted of drilling and installing 3 observation wells at the locations shown on plate 1. The wells were requested by the Arizona Department of Water Resources, Division of Safety of Dams for the purpose of monitoring ground water levels in the project area. The work was accomplished under Contract Modification P00013 at a total negotiated cost of $53,877, and included the drilling, casing, perforating and swabbing of each well by the cable tool drilling method and the subsequent installation of PVC pipe and the placement of a gravel filter. Details of a typical observation well installation are shown in figure 2. Pertinent data for each well are show in tables 3 through 5.

4. Foundation Excavation and Treatment

Dam Foundation

DESCRIPTION

4.01 New River Dam is a compacted, zoned earthfill structure composed of pervious shell zones, transition zones, a central core zone, and a downstream horizontal toe drain (photo 16). The upstream slope is protected by a 12-inch layer of Type I stone up to elevation 1440, an 18-inch layer of Type I stone between elevations 1440 and 1475, and a 24-inch layer of Type I stone between elevation 1475 and the crest of the dam. The downstream slope is covered by 12 inches of Type III stone overlain by landscaping materials. The core and transition zones are founded on bedrock between stations 10+00 and 21+06 and between stations 29+64 and 33+27. The pervious shell are generally founded on bedrock in the abutments, and across the streambed from station 13+00 to approximately station 16+60. The embankment plan, profile, and cross sections are presented on plates 46 through 51.

4.02 The primary objective of New River Dam is flood control. The project is designed to control the standard project flood (SPF) at spillway crest elevation and considering a fully operational outlet works and reservoir pool at spillway crest, the project is designed to safely pass a probable maximum flood (PMF) and provide sufficient embankment and dike freeboard to prevent overtopping by wind-induced wave action.

4.03 The dam embankment was constructed in three stages (photos 17 through 19). Stage I consisted of foundation and core trench excavation from stations 26+50 to 31+90; right (west) abutment excavation from stations 31+90 to 33+27; foundation preparation and treatment, and grouting withi the bedrock core-transition contact zone; and construction of the embankment to elevation 1380 from stations 26+90 to 31+90. Stage II consisted of left (east) abutment excavation from stations 10+00 to 13+00; foundation and core trench excavation from stations 13+00 to 26+50; foundation preparation and treatment, and grouting within the bedrock core-transition contact zone; and construction of the embankment to crest elevation from stations 10+00 to 23+30. Stage III consisted of foundation preparation and treatment within the core-transition contact zone, and construction of the embankment to crest elevation from stations 23+30 to 33+27.

DIVERSION AND CONTROL OF WATER

4.04 The diversion and control of water plan consisted of the construction of two temporary diversion levees to bypass 25-year frequency flows of 28,000 ft^3/sec during Stage I and Stage II construction activities, see plate 52. However, at no time during construction did flood flows disrupt or suspend construction activities due to unusually dry climatic conditions. The Stage I diversion levee (photo 20) was constructed to protect the right abutment, the Stage I foundation and core trench excavations, and the Stage I embankment fill. The Stage II diversion levee (photo 21) was constructed to protect the left abutment, the outlet works, the Stage II foundation and core trench excavations, and the Stage II embankment.

4.05 After grouting the bedrock exposed by the right abutment and Stage I core trench excavations to elevation 1406, the Stage I embankment was constructed to elevation 1380 and capped by a 3-foot protective cover, consisting of a 1-foot lift of erosion resistant silty to clayey sand overlain by 2 feet of spillway rock (photo 22). When the Stage I embankment placement was complete, removal of the Stage I diversion levee began, concurrent with construction of the Stage II diversion levee. This resulted in a 300-foot wide breach between the right abutment and the Stage II levee.

4.06 Once the outlet works was completed and fully operational, closure of the breach commenced in October 1984 during Stage III construction activities. Materials from the Stage II levee were placed in the appropriate zones of the dam embankment during closure. The embankment was constructed to spillway crest elevation 1456 by 27 November 1984, and to elevation 1485 by 31 December 1984. The embankment was topped out in January 1985.

GEOLOGY

4.07 The dam embankment is founded on granitic bedrock of the Precambrian basement complex, Tertiary volcanic bedrock, and Quaternary alluvium. The eastern portion of the embankment, between stations 10+00 and 21+06, is founded on basement complex rock while the western portion of the embankment, between stations 29+64 and 33+27, is founded on volcanic rock. The central portion of the embankment between stations 21+06 and 29+64 is founded on alluvial deposits.

4.08 The geologic conditions at the dam embankment were essentially the same as those anticipated from the preconstruction investigations except that an additional 160 feet of bedrock excavation was required. In the Stage I excavation, the limits of bedrock excavation were established at station 29+64 instead of station 30+28 shown on the contract plans. In the Stage II excavation, the limits were established at station 21+06 in lieu of station 20+10.

4.09 The core trench and abutment excavations were mapped at a scale of 1 inch equals 20 feet using black and white aerial photographs of the excavations as base maps. This proved to be an expeditious and accurate method of mapping bedrock features. The foundation geology for the left and right abutments, and the bedrock portion of the core trench is depicted on plates 29 through 33. Only the most prominent or representative features could be detailed in certain highly complex areas of the foundation because of space limitations. A brief description of the lithologic units encountered during the mapping of the dam foundation (and the other project features) plus a list of the geologic symbols and abbreviations used, can be found on plate 28. The various lithologic units encountered on the east side of the valley are described separately from those on the west side of the valley. A discussion of the geologic structure of the embankment area follows the description of the foundation materials.

Lithologic Units - Left Abutment/Core Trench

4.10 In the original contract plans, the basement complex rocks were collectively referred to as granite. No attempt was made to differentiate by petrographic means the various rock types encountered during the preconstruction geotechnical investigations. Only their appropriate engineering properties were described. However, the fine grained granite and dikes of mafic granite present in certain core holes or test trenches (see plates 13 and 17) can now be referred to as diorite (including quartz diorite) and intrusive rock, respectively. The basement complex was assigned a Precambrian age based upon the Geologic Map of Maricopa County, Arizona (Wilson and others, 1957). During the left abutment and Stage II core trench excavations, a complex assemblage of rock types was exposed. On the dam foundation and outlet works geologic profiles, the granitic rock contacts and other structural features shown on the foundation geology maps were not projected in cross section nor were correlations between core holes attempted because of the complex arrangement of the various rock masses and corresponding lack of detailed subsurface data. This precluded an accurate portrayal of actual subsurface geologic conditions across the eastern portion of the dam embankment except in localized areas. Four basic lithologic units were classified, using both field descriptions and results of laboratory petrographic analyses. A discussion of each rock type is presented in the following paragraphs.

4.11 GRANITE. The dominant rock type present is a light-gray to reddish-brown, medium to coarse grained, phaneritic granite (gr). The rock is characterized by interlocking crystals of quartz, alkali feldspar (chiefly microcline) and plagioclase feldspar (chiefly albite); along with scattered masses of biotite and muscovite. The granite is generally moderately hard to hard, highly fractured, and slightly to moderately weathered.

4.12 DIORITE. The second most common rock type present is a medium to whitish-gray, medium to coarse grained diorite (d). Unlike the granite, the diorite is composed mostly of plagioclase feldspar of andesine composition with only minor amounts of quartz. It also contains a higher percentage of mafic minerals (biotite and hornblende), giving the rock a mottled appearance. The diorite is typically highly fractured like the granite but displays a greater variability in terms of hardness and weathering characteristics.

4.13 QUARTZ DIORITE. Associated with the diorite and possibly representing material near the margins of the diorite pluton which has undergone postmagmatic alteration, is a quartz-rich diorite (qd). The quartz diorite is darker colored and finer grained than the host rock and consists mostly of quartz, alkali and plagioclase feldspars, and small quantities of mafic minerals; including biotite and muscovite. A sample subjected to a petrographic analysis revealed extensive alteration and solution etching of the principal mineral constituents, indicating that post magmatic hydrothermal alteration may have been caused by residual aqueous-gaseous fluids rising through the intergranular pore spaces in the diorite rock mass. The quartz diorite exposures are typically moderately hard to hard, highly fractured, and slightly to moderately weathered.

4.14 INTRUSIVE IGNEOUS ROCK. The basement complex has been intruded in numerous locations by dikes, pods, and lenses of a medium-gray to black (fresh) to reddish-black (weathered), aphanitic, quartz-rich, granitic-type rock (in). The principal constituents of the intrusive rock include quartz, hornblende and biotite with minor amounts of plagioclase feldspar. An intrusive origin rather than a metasomatic (post-depositional mineral replacement) origin is suggested for the dike rock based on texture and the relationship with other lithologies. Petrographic evidence includes: the high amount of quartz, the slight preferred orientation of mineral grains, the small grain size suggesting rapid cooling as compared to the surrounding igneous rock, and the igneous-like texture. The intrusive rock is generally moderately soft to hard, variably weathered, and locally brecciated and rehealed.

Lithologic Units - Right Abutment/Core Trench

4.15 In the original contract plans, it was anticipated that only andesite bedrock would be encountered in the abutment and core trench excavations. However, the tuffaceous agglomerate unit, which was assumed to cap the deeper portion of the bedrock surface beneath the active stream channel, was also found to locally cap the andesite within the limits of the core trench excavation around station 30+00. These volcanic rocks were assigned a Tertiary age based on recent information which suggests that the volcanic rocks in the Phoenix area were probably deposited during the mid-Tertiary orogeny and are at least 20 million years old (Eberly and Stanley, 1978). The andesite and agglomerate were classified using both field descriptions and the results of petrographic analyses. A discussion of both rock types, plus a description of the foundation materials in the alluvial portion of the core trench, is presented in the following paragraphs.

4.16 ANDESITE. The andesite (Tva) flow is characteristically medium to dark gray in color, hard, blocky to platy, moderately to highly fractured, and unweathered to slightly weathered. Macroscopically, the rock has an aphanitic texture with fine spherulitic mafic minerals appearing as black specks. There is also frequent segregation of minute crystals of felsic plagioclase feldspar into discontinuous flow bands with a subparallel alinement. Plagioclase feldspar is the major rock-forming mineral present with the minerals pyroxene and magnetite occurring in only trace amounts.

4.17 Infilling Material. An inspection of the core trench surface, following the completion of Stage I excavation, revealed the presence of a pervasive green clay infilling material along predominantly high angle joint planes in the andesite bedrock. The infilling material was restricted to the core trench bottom between station 29+64 at the edge of bedrock excavation and approximately station 31+70 near the base of the right abutment slope. The green clay is occasionally mixed with a reddish-brown clay and both contain varying amounts of sand to gravel size andesite rock fragments. The infilling tends to be tightly bonded to the fracture surfaces and field evidence (geologic mapping of the core trench and subsurface drilling and grouting) indicated the clay exhibits both lateral and a certain degree of vertical continuity. The origin of this material is questionable. It may represent

alluvial sediments which were deposited into the blocky surface of the andesite flow underlying the valley or may, due to its pervasiveness, represent an in situ weathering product of the buried bedrock.

4.18 Because of the magnitude of the infilling material, various samples were tested to assess its suitability as a foundation material and possible impact on surface preparation procedures. Disturbed, representative samples were collected from the core trench foundation and sent to SPD laboratory as well as to a local laboratory (Earth Technology Corporation). Due to the small size of the samples, only dispersion, soluble salts, and classification tests were performed. The results are inclosed as attachments 3 and 4. Generally, they indicate the infilling material to be a very stiff to hard, nondispersive, erosion resistant plastic clay (CH). Based upon these test results and visual observations, the clay infilling exposed in the core trench was determined to be a suitable foundation material.

4.19 TUFFACEOUS AGGLOMERATE. Although a simple petrographic analysis classified the tuffaceous agglomerate (Tvta) as a lapilli-ash flow tuff, the original rock name used on the contract plans was retained to avoid confusion with a tuff unit exposed in the spillway excavation. This pyroclastic rock is mottled gray to pinkish-red in color, and consists of unsorted, nonstratified, predominantly angular volcanic fragments of andesite and basaltic scoria up to approximately 6 inches in diameter in a vesicular glassy matrix with a pumice-like structure. The matrix contains numerous phenocrysts of plagioclase and alkali feldspar, quartz and biotite. The agglomerate exposed in the core trench excavation is typically massive, moderately soft to moderately hard, and moderately to highly weathered. Field evidence indicates this flow rock forms an approximately 1 to 2-foot cap over the andesite and fills in localized deeper depressions in the underlying bedrock surface. The agglomerate appears similar to that encountered at a depth of 82 feet in preconstruction core hole DD-20 (see pls. 5 and 6) and in fact probably caps the sloping andesite bedrock surface east of station 29+64.

4.20 ALLUVIUM. The central portion of the core trench is founded on alluvium (Qoal), designated as "Stratum C" in the Phase II GDM. The alluvial materials consist mostly of cemented sandy gravels with layers of cobbles and boulders, probably indicative of older stream channel deposits. The results of field and laboratory tests indicated that these materials have high shear strengths, low compressibility and a relatively low permeability and are, therefore, suitable for the dam foundation, including the core.

Geologic Structure

4.21 The project area has been subjected to strong deformational forces from the various tectonic episodes which have affected southwestern Arizona throughout geologic time. Numerous shears and shear zones, fractures, alteration zones and igneous intrusions have significantly affected the Precambrian basement complex rock. In the volcanic andesite rock, most of the structural features like layering and jointing probably formed during movement and solidification of the lava flow. The apparent oversteepened contacts of the layered volcanic sequence in the spillway and the orientation of the layering suggest possible uplift and tilting subsequent to emplacement. The

structural trends in the mountains bordering the project area are predominantly to the northwest, conforming with the dominant pattern noted in the Salt River Valley area by McDonald and others (1947). Even the long axes of the mountain ranges themselves reflect this trend. However, the shearing prevalent in the granitic rocks tends to be oriented in a northeast direction.

4.22 The geologic structure of the dam foundation was determined mainly from surface exposures, supplemented by subsurface bore hole and geophysical data. The actual bedrock profile revealed during construction was essentially the same as depicted on plate 5 on the contract plans. Except for the extension of the bedrock pediments on both sides of the valley, the only other significantly different foundation condition encountered was the abrupt change in bedrock gradient between stations 16+40 and 16+90. As originally envisioned, the depth to bedrock was anticipated to increase gradually from a depth of approximately 6 feet at station 16+00 to a depth of about 23 feet at station 19+00. However, the steep 14-foot high slope present instead may represent a possible erosion scarp developed in the softer predominantly diorite bedrock by a pre-existing river channel (photo 23). Evidence for an ancient river channel condition includes: (1) layers of stream deposited cobbles and boulders present in the thicker sections of alluvium west of the bedrock slope, and (2) extensive scouring and water-rounding of the bedrock surface underlying these older stream deposits (photo 24). However, only several small isolated scour channels were noticeable in the foundation surface adjacent to the slope, probably due to the softer, more weathered nature of the bedrock.

4.23 The contact between the Tertiary andesite flow and the Precambrian basement complex probably occurs at depth beneath the active stream channel as shown on plate 5. The intense tectonic upheaval characteristic of the Laramide orogeny suggests that this contact probably represents an erosional unconformity. The basement rock has undoubtedly been altered by contact metamorphism resulting from emplacement of the volcanics. The existence and position of subsurface basement (or boundary) faults beneath the project has never been established due to the lack of deep subsurface information, and the absence of evidence for faulting at or near the ground surface. If such large-scale faults do exist, they would most likely be manifestations of the long-quiescent Basin and Range disturbance. The predominance of non-tectonic landforms, such as extensively pedimented mountain blocks, in southwestern Arizona suggests that active block-faulting ceased during the late Miocene or early Pliocene (Pearthree and others, 1983), and therefore, any basement faults would be considered inactive. Of all the small-scale shears and faults found in trench excavations, none were observed to displace the overlying Quaternary alluvial deposits.

4.24 The foundation bedrock exposed in the left abutment and Stage II core trench excavations consists of basically two large, highly fractured and sheared plutonic bodies ranging from granite to diorite in composition. Unlike the relatively uniform physical characteristics of the volcanic rocks, the basement complex is quite variable in terms of hardness and degree of weathering. The granite tends to be the most competent of the basement rocks. Throughout the excavation, exposures of granite are consistently moderately hard to hard, and slightly to moderately weathered. The diorite, on the other

hand, is more susceptible to weathering, probably because of its quartz-deficient composition. As a result, its physical properties are usually more variable. On the left abutment and in the vicinity of the outlet works (pls. 32 and 33), the diorite is generally moderately hard to hard and slightly to moderately weathered. However, in the area of the bedrock slope between stations 16+50 and 17+50 (pl. 31), the bedrock exposures are normally moderately soft to moderately hard and moderately weathered. The hardness and weathering characteristics of the quartz-rich diorite tend to be similar to those of the granite.

4.25 The granite and diorite plutons appear to have been invaded in numerous places by dikes, pods, and lenses of a quartz-rich, mafic igneous rock. From station 13+50 to station 17+50, the intrusive rock generally occurs as tabular dike-like bodies (photo 25) which display a persistent northeast trend, almost parallel to that of the major shear zone (see pls. 31 and 32). However, on the left abutment (pl. 33), the trend changes to a more northerly direction and irregularly pods and lenses are more obvious. As stated previously, an intrusive origin for the dike rock is suggested based on field and petrographic evidence. The intrusive rock cuts across the structure of both major rock types but overall appears to be more closely associated with the diorite. In fact, when in contact with the granite, it tends to form the boundary between the granite and adjacent dioritic rocks. The dike rock was assigned a Precambrian age but might possibly be a product of the Laramide orogeny, which affected the mountains to the east of the project. Exposures of this lithologic unit are generally moderately soft to moderately hard, slightly to highly-weathered, and highly fractured to shattered. The rock is locally brecciated and rehealed with clay gouge and in some instances, appears to have undergone partial metamorphism. This deformation may be related to subsequent tectonic episodes which induced shearing of the basement rock. The intrusive dikes generally vary from 1 to 10 feet in width. However, a prominent "V"-shaped exposure of soft, highly weathered intrusive rock, which trends in a northeast direction oblique to the dam axis, attains a maximum thickness of approximately 25 feet upstream of station 14+90 (see pl. 32). When this localized and rather unique subsurface condition was encountered during the core trench excavation, the large tracked backhoe was able to easily create a 12 to 14-foot deep, 30 by 40-foot wide depression (photo 26) in the foundation surface before excavation was halted. This depression was situated 5 feet upstream of the dam centerline between stations 14+85 and 15+25 (see pls. 32 and 47). The excavation occurred along a postulated near-vertical contact between soft, weathered intrusive and diorite rock. The excavator did not encounter resistance sufficient to terminate excavation because of the fairly consistent subsurface conditions present. Despite the greater depth of excavation, the physical characteristics of the bedrock in the bottom of the depression were not significantly different from those at the surface.

4.26 The basement complex has been extensively dissected in places by a prominent shear zone and a secondary network of smaller individual parallel to cross shears. The shearing displays a dominant northeast structural trend, compared to the major northwest trend of the region. Measured shear attitudes, plotted on a stereonet, revealed an average strike of N70°E and a dip of 75°SE. The major shear zone (see pls. 30 through 33) generally

parallels the dam alinement. It is confined mostly to the upstream transition zone of the core trench and left abutment (photos 23 and 27), except in the vicinity of the outlet works between stations 13+65 and 15+00. There it exits the core trench excavation and apparently re-emerges in the outlet conduit trench excavation as a narrow 12-inch-wide zone at station 19+05. The shear zone displays a characteristic diversity in structure. Between stations 10+00 and 19+00, the zone is noted for its apparent sinuosity and variable thickness. It ranges from approximately 4 inches to 20 feet in width and exhibits little linearity, having the tendency to warp, pinch and swell. The shearing generally cuts across the structure of all the major rock types but also occasionally wraps around bodies of igneous rock without dissecting them. An example of this phenomenon occurs upstream of station 12+10 where the southern edge of the shear zone "curves" around a lense of intrusive rock (photo 28). West of station 19+00, the shear zone branches out into two distinct parallel linear shear zones averaging 4 inches in width with a more east-west trend. Significant shearing within this portion of the excavation also tends to be absent. Numerous individual shears which branch or splay out in divergent patterns from the major shear zone are quite common throughout the excavation (photo 29). These shears range in thickness from a pencil line to several feet and few are continuous in length for more than 200 feet. These shears also have characteristic non-linear patterns. On the left abutment slope, several approximately 5-foot wide zones containing calichified sheared rock rubble extend diagonally across the foundation surface from the shear zone.

4.27 The major shear zone contains basically a mixture of crushed and brecciated to intensely fractured rock and clay gouge, with some evidence of slickensides. The rock has variable hardness and weathering characteristics and frequently is unrecognizable in terms of rock type due to the effects of shearing and/or hydrothermal alteration. Shear-induced block rotation has resulted in locally crushed to shattered areas in rock masses adjacent to the shear zone. This type of deformation was usually confined to the left abutment. The fractured basement rock in the shear zone and other portions of the excavation, particularly on the left abutment and shallow pediment surface in the core trench, has been extensively rehealed with pervasive secondary deposits of calcium carbonate, primarily in the form of indurated insoluble caliche (photo 30). Calcium carbonate lined fractures up to 3 inches wide are most prominent within the diorite rock mass, particularly on the left abutment.

4.28 Joint structures in the basement complex are quite extensive and well-developed (photo 31). They are usually tight or well healed (generally with calcium carbonate), and typically closely spaced (1/2 to 6 inches). Open or clay-lined joints were relatively few, and their apertures were too narrow to permit successful slurry grouting. Strong subparallel to parallel patterns are recognizable in the joint arrangements, particularly those with a northwest trend. Joint orientations are variable but three distinct, although broadly defined, patterns emerged when the attitudes measured in the excavation were plotted on a stereonet. The stereonet plot revealed what may be referred to as orthogonal joint sets, which are characteristic of block-jointed granite. The major joint systems (with average strikes and dips noted) are as follows: (1) striking N55^{o}E, dipping 70^{o}SE; (2) striking N25^{o}W,

dipping 55°SW; and (3) striking N40°W, dipping 50°NE. The origin of the jointing is debatable, but it is believed that shearing is not the only mechanism responsible for the highly fractured nature of the igneous rock mass. The first joint system lies at an acute angle to the dominant shear direction and thus may be an "en echelon" feature caused by shearing. The remaining two joint systems may have been formed by processes relating to mid-Tertiary volcanism in the project area. The hypothetical development of these northeast trending systems with opposite dip directions may be as follows. If silicic magma began accumulating in a shallow magma chamber, the resulting pressure build up would cause the granitic basement rock above the chamber to arch upward. Intense mechanical fracturing would result due to this arching effect, giving the magma a pathway to the surface. Since this venting obviously occurred somewhere on the west side of the project, the east side bedrock would have healed due to the release of pressure.

4.29 No definitive faulting was identified in the left abutment and core trench excavations, and the shearing is believed to be contemporaneous with the large-scale mid-Tertiary orogeny. The shearing is probably not related to emplacement of the Precambrian basement complex since it was found to cut across all bodies of igneous rock, including the intrusive dike rock.

4.30 The limited exposure afforded by the dam excavation, the relative complexity of the igneous rock assemblage, and the lack of conclusive field evidence or radiometric age dates made any attempt at determining the exact relationship between the various rock units tenuous at best. Unlike the relatively simple layered volcanic sequence in the spillway, the lithologic units comprising the basement complex frequently occur as irregularly-shaped, discontinuous masses without any significant lateral continuity or dominant structural trend. A good example of this occurs on the left abutment between stations 10+00 and 12+00. It is not known whether emplacement of the granite and diorite plutons was contemporaneous or occurred at different times. Apparent xenoliths of both rock types appear to be mutually occurring, while the existence of the quartz diorite might indicate hydrothermal alteration of the diorite pluton during intrusion of the granite magma (or for that matter the intrusive rock). If this latter hypothesis is true, then the so-called Precambrian age granite may in fact be younger rock, possibly related to the Laramide orogeny.

4.31 The foundation bedrock exposed in the right abutment and Stage I core trench excavations is predominantly andesite with only minor amounts of tuffaceous agglomerate present. The andesite is strongly jointed and has a well-developed blocky structure (photo 32). The viscous nature of the flow during emplacement probably resulted in the lava developing its distinctive structure as it cooled. The andesite also exhibits a generally well-defined layering, which dips in an upstream direction and gives the rock a locally platy or slabby appearance (photo 33). The layering may be the consequence of differential cooling of a single lava flow or the eruption of numerous thin flow sheets. However, evidence of shearing, indicating possible movement of one layer over another, was difficult to recognize. The only obvious flow structures consist of oriented phenocrysts of plagioclase feldspar into discontinuous flow lineations or bands. The irregularity of the foundation surface can be attributed in part to the hard blocky nature of the andesite rock mass.

4.32 Joint structures are typically well-developed, open, closely to moderately spaced (1 to 12 inches), and arranged in subparallel to wedge shaped patterns with varying degrees of lateral continuity. Joint orientations are variable, although strong consistent patterns are recognizable throughout the excavations and in stereonet plots. The dominant trend in the layering is N50°W, with an average dip of 30°NE. Two high angle intersecting joint sets predominate; one striking N70°W and dipping 60°SW and the other striking N20°E, dipping 75°NW. However, both attitudes are averaged within two broadly defined patterns indicated on stereonet plots. Unlike the usually tight or narrow calcium carbonate healed joint apertures present in the granitic rock, the characteristically open joints in the volcanic bedrock are generally lined with varying thicknesses of infilling materials. In the core trench bottom, green clay (occasionally mixed with red-brown clay) filling of discontinuities with apertures averaging approximately 1/4 to 2-1/2 inches in width is quite extensive. On the abutment, similiar discontinuities are usually filled with fine grained residual soil or rock rubble, although minor calcium carbonate and black oxide staining are also present. Scattered throughout the excavations are brecciated and rehealed fractures, frequently interconnected, which occasionally extend uninterrupted for distances of up to 50 feet. The well indurated breccia, which can attain a maximum thickness of about 12 inches, is composed mostly of purplish-gray andesite (photo 34). However, minor amounts of reddish-brown flow breccia are also present. Joint surfaces are typically planar to slightly undulant in shape although distinctly arcuate patterns are prominent in localized portions of the core trench. Voids and near vertical linear zones of rock rubble and soil occasionally separate intact coherent rock blocks on the abutment surface. These surface features, ranging from approximately 1 to 3 feet in width and 2 to 4 feet in depth required cleaning and backfilling with dental concrete prior to embankment placement. In addition, joints and fractures containing unsuitable foundation materials, primarily loose rock and soil, were cleaned and treated with grout slurry to provide a suitable foundation.

4.33 The andesite rock mass, as noted previously, is generally moderately to highly-fractured. However, in localized areas; particularly on the upper half of the abutment and near the margins of the core trench bedrock excavation east of station 30+20, the rock is highly fractured to shattered. The distinctive layered structure is also not as apparent within portions of the upper half of the abutment. Instead, the foundation surface has the appearance of large randomly oriented volcanic blocks separated by predominantly northeast trending near vertical joints and rubble zones. These areas may in fact represent the more intensely fractured near surface or leading edge of a highly viscous flow.

4.34 The only exposures of tuffaceous agglomerate occur between stations 30+15 and 29+64, an area where only limited excavation of the andesite bedrock was accomplished due to the increasingly greater depth to bedrock toward the active streambed. Outcrops are sporadic due to the cap-like nature of the rock unit. In addition to the agglomerate, the andesite upstream of station 30+00 is also capped by a layer of andesite breccia in a matrix of green clay and fine grained, tan colored tuffaceous material.

FOUNDATION EXCAVATION

4.35 Foundation excavation consisted of excavating the near surface alluvial materials from beneath the entire dam embankment down to bedrock or elevation 1380. The depth of foundation excavation is shown on plate 46. The purpose of the foundation excavation was to remove unsuitable materials (primarily silty sands) which would be susceptible to collapse when loaded and saturated. The removal of these materials would minimize differential settlement and provide a suitable foundation (either bedrock or a sandy gravel layer) on which to place the pervious shell materials. Bedrock was encountered on the east side of the valley between station 13+00 and approximately station 16+60 beneath the pervious shell zones. Most of the excavation was accomplished using push dozers and scrapers. However, when caliche cemented alluvial materials were encountered above the shallow bedrock pediment on the east side, a D8K dozer equipped with a double shank cross ripped the material to facilitate removal of the alluvium with the scrapers (photo 35).

ABUTMENT/CORE TRENCH EXCAVATION

4.36 The purpose of the abutment and core trench excavations within the core-transition contact zones was to provide a suitable foundation on which to place and compact core and transition materials. In this regard, the excavations were extended either to sound bedrock (photos 36 and 37), or in the case of the central portion of the dam foundation where bedrock was deep, to a well cemented sandy gravel layer at elevation 1365. Within the pervious shell zones on the abutments, only minimal excavation (1 foot average) was required to remove all loose overburden and rock, vegetation and other objectionable materials. The depth of the abutment and core trench excavations are shown on plate 46.

Right Abutment/Stage I Core Trench

4.37 ABUTMENT ACCESS ROAD. The excavation for the right (west) abutment and Stage I core trench commenced on 17 November 1983 and was completed by 5 January 1984. Excavation of the steep right abutment was probably one of the most difficult and potentially hazardous phases of the entire construction process. To effect equipment access to the abutment prior to the start of excavation, the Contractor, starting on 27 October 1983, pioneered a side hill cut access road from the miscellaneous fill area up the right abutment slope outside the abutment-embankment contact zone to a staging area about 15 feet above the crest of the dam. The initial road excavation was accomplished using a D9H dozer (photo 38). Due to difficulties in ripping hard resistant outcrops of volcanic bedrock, (i.e., andesite and flow breccia), only 150 feet of road was excavated after three work shifts. The Contractor then proposed to implement a "secondary type" blasting program to assist their pioneering efforts along the remaining 500-plus feet of roadway and to preclude further delays in the tight Stage I construction schedule. Two production shots, on 8 and 11 November 1983, were necessary to blast the bedrock along the road alinement while a third shot, conducted on 15 November, was necessary to widen the perimeter of the abutment staging platform to allow for greater equipment mobility and safer working conditions. According to the blasting summary in table 6, 552 cubic yards of rock were blasted using 430 pounds of Atlas 75-

percent Power Primer explosive. This resulted in an average powder factor of 0.78 pounds of explosive per cubic yard of rock. Blast holes were drilled on a 4 x 5-foot pattern, to depths ranging from 5 to 7 feet along the access road to 8 to 10 feet within the staging area. The muck generated from the 3 production shots was pushed down the slopes using a D8 dozer. The access road blasting program afforded the Contractor the opportunity to establish acceptable shot reporting and recording procedures before commencing blasting operations in the spillway.

4.38 ABUTMENT EXCAVATION. Once the right abutment access road was completed, the Contractor field tested his proposed abutment excavation technique on the spillway muck pile. The demonstration worked well although subsequent changes were required during abutment excavation to produce a safer and more effective operation. The Contractor initially attempted to strip the loose rubble from the abutment surface using a small Case 1150C dozer attached by a single winch cable to a D8H dozer on the staging platform. By winching the small dozer up and down the slope, swaths could be cut in the loose material using the blade. However, this technique proved unsucessful and was soon abandoned. The main problem was that if the Case dozer was not directly in line with the drum on the D8 dozer, the winch cable could not be reeled up properly. This meant that the D8 had to constantly shift around to maintain proper alinement with the Case dozer as it worked the slope. This excessive movement caused much sloughing of the bedrock at the edge of the platform, creating a safety hazard for the dozer operator working below. In addition, the winch cable was becoming increasingly frayed due to contact with the bedrock surface.

4.39 On 21 November, the Contractor resumed the right abutment stripping using the Case dozer attached by the winch cable to two D8 dozers moving in tandem along the staging platform (photo 39). This modified procedure was successful in blading loose rock and overburden from the abutment surface within the core-transition contact zone and upstream pervious shell zone but was not practical for accomplishing the approximate 3 feet of bedrock excavation anticipated for the abutment core and transition zones. Due to the inability of the dozer to develop sufficient leverage on the steep slope, the blade had limited success in removing the typically loosely keyed in-place andesite rock blocks from the abutment surface (photo 40). The Contractor then attempted to reach a suitable foundation surface using both high and low pressure air blasting followed by hand labor but it soon becaome apparent these methods were both time consuming and ineffective. Detailed cleaning of selected test areas on the upper half of the abutment to allow visual inspection and evaluation by Geotechnical Branch personnel of the foundation surface revealed unsuitable materials, generally loose, soil infilled rock blocks, which could be further excavated up to depths of about 2 feet using a rock pick.

4.40 Although reservations were expressed by the Government over the affects of ripping on the fractured bedrock, it was obvious that mechanical methods would be necessary to accomplish the anticipated 3 feet of abutment excavation as shown on the contract plans. The Contractor then decided to abort the cleaning program and resume excavation on 2 December using a D9H dozer with a double shank to rip the lower half of the abutment (photo 41). The dozer was winched up the slope by another D8H dozer using a D9H dozer as a "deadman".

This set-up worked well except the winch periodically overheated and the D8 dozer could be pulled only half way up the slope; the remainder was too steep. The upper ±3 feet was ripped fairly easily and the muck included a significant amount of alluvium.

4.41 On 5 December, the Contractor switched back to the Case 1150C dozer, this time equipped with a triple-toothed ripper (later changed to a double tooth ripper), to rip the upper half of the abutment as well as complete the excavation of the lower portion (photo 42). The lighter weight dozer was ideally suited for excavating the 2 to 4 feet of fractured and infilled andesite in the core-transition contact zone. Initially the same overheating problems were encountered but after switching to a double shieved winch cable, no further difficulties were incurred. The problems of working on a steep slope were partially solved by pushing the muck pile at the base of the abutment up the slope to create a berm with a flatter gradient. When refusal was encountered, several test areas were air cleaned to allow visual inspection and evaluation of the foundation surface. The bedrock looked treatable, although extensive foundation preparation would be necessary in localized areas to remove loose rock blocks, and infilling material (photo 43). A comparison with the andesite bedrock exposed in the spillway and staging area indicated that additional mechanical excavation would probably not expose a significantly better foundation surface and that due to the nature of the joint patterns and the limitations imposed by the steep abutment surface, additional excavation in refusal areas would probably be difficult, ineffective, and unnecessary. Therefore, the Contractor was instructed that whatever the dozer could effectively remove by ripping and blading would be the extent of abutment excavation.

4.42 CORE TRENCH EXCAVATION. Once the above streambed excavation was completed, and the talus pile at the base of the abutment removed, the Contractor, on 16 December, began the Stage I core trench (below streambed) excavation using two scrapers, assisted by a D9H push dozer and a D8K dozer equipped with a double shank. The D8 dozer excavated the bedrock on the abutment slope using the rippers and blade, while the scrapers worked at removing the ripped bedrock and alluvial materials within the core trench excavation (photo 44). The removal of the talus pile generated by the abutment excavation revealed an approximate 5-foot gap between where the D8 and Case dozers stopped excavating and the D8 dozer began its excavation. This gap was subsequently excavated when the scraper work was completed.

4.43 By 20 December, andesite bedrock had been encountered in the bottom of the core trench so scraper use was discontinued. It was apparent that near the base of the abutment slope, more than 3 feet of bedrock had been excavated, and that between approximately stations 31+50 and 30+28, a gradual slope from elevation 1355± up to elevation 1365 had been established. A cursory inspection of the bedrock exposed by the scrapers indicated that a treatable foundation surface had not been reached, even in areas where approximately 3 feet of bedrock excavation had been accomplished. To avoid undue damage to or degradation of the highly fractured bedrock, the Contractor used a large tracked backhoe (Caterpillar 235 excavator) instead of a dozer with rippers to continue excavation (photo 45). Due to Government concerns that the "break-out" power of the excavator might result in significant over

excavation, the Contractor was instructed to use the following criterion to define "refusal". Once the bucket teeth scratched the rock surface without loosening or removing large amounts of material, but instead leaving noticeable streak marks, then excavation would be discontinued. This method of establishing a treatable foundation surface worked well except that the average depth of excavation in the bottom of the core trench was approximately 7 feet below the estimated depth shown on the contract plans. On the lower abutment slope which the D8K dozer had initially excavated, the excavator was able to remove an additional 1 to 2 feet of bedrock and was also able to work the 5 foot section untouched during previous excavation attempts.

4.44 As the excavation approached station 30+28, which was the original estimated limit of Stage I bedrock excavation, the decision was made to "chase" bedrock out to the point where it dropped below the existing average base elevation of 1355. A maximum 5-foot deep exploratory trench was dug with the excavator parallel to the dam axis between approximately stations 30+20 and 29+55 (see pl. 29). The trench exposed a somewhat undulating andesite bedrock surface extending out to station 29+64 before dropping below invert grade. Depressions in the bedrock surface were filled with a material classified as a tuffaceous agglomerate.

4.45 Once the limits of bedrock excavation in the core trench had been established, the remaining bedrock was exposed using a Case 680G backhoe equipped with a scraper bar welded across the bucket teeth. However, it soon became apparent that the existing core trench, due to the greater depth of bedrock excavation was now too narrow to accommodate the full width of the core and transition zones. The decision was made to construct the dam embankment as per design so between stations 29+10 and 31+92 the core trench was widened to accept the increased width of the core zone plus the two 15-foot wide transition zones (see pl. 51). The side slopes of the trench now varied from 1V on 1H in rock to 1V on 1.5H in alluvium. In addition, the transition from top of bedrock (with a 4H to 1V slope) to the projected top of the base of transition was established at station 29+64 in lieu of station 30+28. The additional Stage I excavation, which required the removal of 13,685 cubic yards of material to provide an acceptable bedrock foundation within the core trench, was handled under Modification of Contract P00005 at a total negotiated cost of $68,227. To avoid degradation of the cleaned foundation surface, scrapers cut benches in the alluvial side slopes so the excavator could set up and reach the base of the excavation and begin widening the core trench (photo 46). The excavated material was loaded into end dumps for eventual disposal in the miscellaneous fill area. The additional work commenced around 3 January and was expeditiously completed by 5 January 1984.

Left Abutment/Stage II Core Trench

4.46 The excavation for the left (east) abutment and Stage II core trench commenced on 22 November 1983 and because of several long delays caused by conflicts with other construction activities or restrictions imposed by the contract specifications, was not completed until 23 March 1984. The contract plans called for approximately 5 feet of abutment excavation and about 3 feet of core trench excavation. Prior to the start of excavation within the core-transition contact zone, the Contractor "stripped" the entire left abutment

using a D9H dozer to a depth of approximately 1 to 2 feet. The purpose of this excavation was to provide a fairly even uniform working surface within the core-transition contact zone and to remove objectionable materials, namely loose rock and vegetation, within the pervious shell zones. Once this initial stripping was completed, no further work was done on the left abutment until 5 January 1984 as the Contractor concentrated his efforts on the right abutment and Stage I core trench excavations.

4.47 Before the Stage II excavation resumed, a meeting was held between Government and Contractor representatives to discuss bedrock excavation methods. The Contractor was reminded that heavy tracked equipment would not be allowed on the foundation surface to preclude degradation of the highly fractured and weathered granitic bedrock at invert grade. Mechanical excavation by ripping would be limited to a depth of approximately 4 feet to preclude damage to the underlying bedrock. The Contractor was also instructed to exercise tight survey control during the progress of the work, so necessary adjustments in the base width of the abutment or core trench excavations could be made during excavation and not at the completion of the work.

4.48 ABUTMENT EXCAVATION. The Contractor elected to use both a dozer and the 235 excavator to excavate the left abutment. The dozer would initially rip the upper ±4 feet of material in 15-foot sections within the core-transition contact zone and then push the muck downslope to build a berm for the excavator. The excavator would then set up and scrap off the remaining ±1 foot of material down to a treatable bedrock surface (photo 47). In this way, heavy tracked equipment would not come in direct contact with the abutment surface. The criterion used for establishing "refusal" for the Stage I core trench excavation was again followed. To expedite the first phase of the Stage II excavation program, a scraper bar was welded across the excavator's bucket teeth so that the remaining bedrock excavation and initial cleaning could be accomplished concurrently, followed by more detailed air blasting.

4.49 CORE TRENCH EXCAVATION. Once the abutment excavation was completed, bedrock excavation in the core trench between the left abutment and station 16+50 was accomplished using only the excavator. Due to an intensive effort on the part of the Contractor, the first phase of Stage II excavation within the core-transition contact zone, which commenced on 5 January 1984 was completed by 13 January 1984. Station 16+50 was established as the approximate limit of the initial excavation because: (1) beyond that point, the bedrock surface appeared to drop off rather abruptly, which would have required an extensive amount of alluvial excavation, and (2) since the Stage II diversion levee had not yet been completed, taking the core trench excavation progressively deeper would have posed problems from a safety standpoint.

4.50 The excavator produced a fairly uniform level surface, except for localized areas of irregular blocky bedrock on the left abutment, and a tolerance for excavation which in most cases did not vary significantly, given the nature of the bedrock, from that shown on the contract plans. Between approximately stations 13+25 and 15+25, however, excavation in bedrock had to be carried through to depths ranging from 6 to 14 feet to reach a suitable foundation surface. In only one localized area did the excavator remove a

large quantity of material without ever encountering sufficient resistance to terminate excavation. Upstream of station 15+00, a 12 to 14-foot deep, 30 by 40-foot wide depression was excavated in soft, weathered diorite and intrusive rock before work was halted (see pl. 47). An inspection of the bedrock depression indicated that the bedrock surface would provide a suitable foundation and further excavation was discontinued.

4.51 Once the Stage I embankment fill placement was completed, allowing removal of the Stage I diversion levee concurrent with resumption of the Stage II diversion levee construction, Stage II excavation resumed on or about 1 March 1984. Removal of alluvial materials between stations 16+50 and 26+50 was accomplished using push dozers and scrapers (photo 48). When the top of bedrock was encountered in the core trench excavation, scraper use was discontinued and the bedrock surface fully exposed using up to 3 Case backhoes equipped with scraper bars in lieu of the excavator. A D9H dozer assisted the scrapers during excavation by ripping the cemented alluvial deposits above top of rock to facilitate their removal. The dozer also trimmed the 1V on 1.5H alluvial cut slopes to grade using a slopeboard and adjusted the base width of the core trench when necessary to conform to the top of bedrock elevation. However, all final slope and base widtn adjustments prior to embankment fill placement were made using a large Drott tracked backhoe equipped with scraper bar. Only minimal bedrock excavation was required west of station 17+50 to remove weathered less competent rock and provide a suitable foundation for grouting and foundation treatment. The second phase of the Stage II excavation revealed the following: (1) the bedrock surface did not maintain the gentle gradient depicted on plates 5 and 46 of the contract plans, but instead exhibited a rather abrupt 15-foot drop in elevation between stations 16+40 and 16+90; and (2) bedrock was exposed at invert grade (approximate elevation, 1365 feet) between station 20+10, the original estimated limit of bedrock excavation, and station 21+06. Most of the alluvium near the steep bedrock slope was excavated using a Caterpillar 966 front end loader because the area was inaccessible by scraper (photo 48). The limits of Stage II bedrock excavation were firmly established by digging several shallow exploratory trenches with a Case backhoe parallel to the dam axis.

FOUNDATION PREPARATION AND TREATMENT

General

4.52 The bedrock foundation preparation and treatment consisted generally of the following: (1) cleaning, (2) dental concrete, (3) grout slurry, and (4) subsurface pressure grouting. The purposes of bedrock surface preparation and treatment within the core-transition contact zone of the dam and dike no. 1 embankments were to: (1) seal all open joints and fractures in the core zone rock to prevent the embankment materials from being piped into the rock openings, and (2) prepare surfaces to achieve satisfactory contact with the overlying compacted embankment materials. The grouting was conducted primarily to explore the subsurface foundation conditions and to create a relatively impermeable zone under the core zone of the dam. The main objective in the preparation and treatment of bedrock surfaces within the outlet works and spillway where structural concrete was placed was to insure that an adequate bond was established between the concrete and rock

foundation. The above foundation preparation and treatment was not required for dike no. 2 and the pervious shell zones of the dam and dike no. 1 embankments because the coarse nature of the pervious shell materials would preclude their piping into openings in the underlying bedrock.

4.53 Before surface treatment or placement of embankment materials or structural concrete on any part of the bedrock foundation surface, such areas required inspection and approval by the Government, primarily Engineering Division, Geotechnical Branch personnel. Approval was done in sections, the limits of which generally depended on such factors as the Contractor's construction schedule and the effect of bedrock topography on the rate of material placement. For example, extensive sections of the core trench could be approved at one time because of the more rapid placement of embankment material over the relatively flat bedrock surface. Foundation approval on the steeper abutment slopes was generally restricted to approximately 10-foot increments because of the slower rate of fill placement against the slopes.

4.54 Foundation treatment (dental concrete and/or grout slurry) was accomplished just prior to embankment fill or structural concrete placement. However, foundation preparation was required prior to the following construction activities: (1) foundation drilling and grouting, and/or foundation geologic mapping; (2) surface treatment; and (3) placement of embankment materials or structural concrete. The same criteria for foundation suitability were followed during each stage of construction. In general, the specifications required that all loose rock and other unsuitable materials (those deemed incompatible with the overlying fill or concrete) be removed from all bedrock surfaces within the core-transition contact zone or upon which concrete was to be placed. In addition, bedrock discontinuities such as joints and fractures containing unsuitable infilling material were to be cleaned to a depth of three times the structure width. Similar methods were employed to achieve acceptable foundation surfaces. However, due to varying or sometimes unique foundation conditions, changes were frequently necessary to achieve the desired results.

4.55 For portions of the dam and dike no. 1 embankments founded on alluvial materials, foundation preparation procedures were as follows. Upon completion of the required excavation, the invert surface (including core trench and exploration trench sidewalls) was inspected to insure compatibility of the embankment material with the foundation materials. Sections were then approved for fill placement by Government personnel. Prior to placement, each section was scarified to a depth of 6 inches and compacted with 8 passes of a 50-ton articulating rubber tired roller.

4.56 Since foundation preparation was anticipated to be a lengthy, labor-intensive process, a separate bid item for this activity was included in the contract bidding schedule. As originally planned, measurement of foundation preparation was to be based solely on the number of man-hours required to acceptably complete the work using hand methods only. However, it soon became apparent that the use of certain mechanical equipment, particularly small backhoes, to expose the bedrock surface during the initial stages of foundation preparation helped facilitate subsequent cleaning using hand methods and air blasting. Since the use of such equipment proved to be

beneficial as well as advantageous to the foundation cleaning program, the original estimated payment item quantity was modified to include those equipment-hours (expressed in equivalent man-hours) required for foundation preparation.

Right Abutment/Core Trench

4.57 Preliminary surface preparation of the right abutment during Stage I construction activities was the most risky and time-consuming part of the entire cleaning process. The potential hazards involved when working on a steep, irregular and occasionally wet slope led to the adoption of several extraordinary safety measures. All personnel working on the abutment were required to be tied to ropes or "safety lines" (photo 49) due to the rather precarious footholds. A man basket was stationed on the staging platform, with sufficient rope to lower an injured person, should an accident occur, to the bottom of the abutment. Laborers working in close proximity were instructed to remain at the same level to avoid the hazard of falling rocks. In addition, the use of a 2-inch bull hose for high pressure air blasting generally required two hose tenders instead of one to assist the nozzleman. Despite the less than ideal working conditions on the right abutment, no serious injuries resulted during this phase, or for that matter, any phase of foundation preparation.

4.58 ABUTMENT - PRELIMINARY SURFACE PREPARATION. Once the abutment excavation was completed, the Contractor cleaned the core-transition contact zone using both high and low-pressure air blasting and hand labor (photo 50). The abutment surface was initially given a rapid air cleaning to remove most of the loose rubble generated from the abutment excavation (photo 51). Once this was done, the foundation rock was subjected to several detailed cleanings to prepare the surface for grouting. In general, the laborers worked from the top of the abutment downslope to avoid contaminating previously cleaned areas. Since the andesite bedrock was highly fractured, blocky and easily degradable, characteristics which did not change significantly with depth, foundation preparation had to be done carefully to avoid removing excess rock. Therefore, certain procedures were established to maximize the effectiveness of the detailed cleaning program. Each area was first cleaned thoroughly using high-pressure air blasting. Then, additional loose rock fragments or blocks, organics, and soft alluvial infilling material between the andesite blocks were removed by hand and with rock picks. The laborers were instructed to test the competency of any suspect rock by lightly tapping it with a rock hammer before attempting to remove it. This was done to discourage the use of the rock pick as a pry bar to dig out suitable rock. Once the detailed hand labor was finished, each area was then re-blown with a smaller diameter (approximately 1-inch) low pressure air hose to avoid dislodging and loosening up additional bedrock. Rock chipping guns were used for a limited period of time to facilitate the removal of large rock blocks but their use was discontinued when it became evident that the guns were doing more damage to the rock foundation by propagating existing fractures and creating new ones.

4.59 CORE TRENCH - SURFACE PREPARATION. The Stage I core trench bottom was cleaned fairly rapidly utilizing up to two Case backhoes equipped with scraper bars, shovels, rock picks, and high pressure air blasting. The pervasive

green clay fracture filling encountered in the core trench excavation essentially reduced foundation preparation procedures to removing loose rock and other objectionable materials from the bedrock surface. However, joints and fractures were "raked" with rock picks as necessary to loosen dessicated clay infilling material and expose competent material. Cleaning to a depth of 3 times the fracture width, as required by the contract specifications, was not necessary since the clay had been determined to be suitable foundation material. In general, surface preparation followed the same pattern as the core trench excavation, progressing out toward station 29+64. The initial cleanup was accomplished by mechanical equipment and hand labor (photo 52). Most of the muck generated by the surface cleanup was carried by backhoe to a front end loader for loading into a rock truck and eventual disposal in the miscellaneous fill area. Any remaining surface material was subsequently removed by air blasting. Once the preliminary cleanup of the core trench was completed on 9 January 1984, foundation drilling and grouting operations commenced the following day.

4.60 Foundation preparation of the Stage I core trench surface resumed on 6 February 1984, after conclusion of all required grouting and geologic mapping activities. Only hand methods and air blasting were required during this cleanup phase; mechanical equipment was not necessary. The purpose of this cleanup was to prepare the bedrock surface for any foundation treatment and subsequent embankment fill placement. In this regard, debris and waste from the grouting operations, including drill cuttings, grout, and rock loosened or damaged by the air track drill; and loose rock fragments, dried clay infilling, and other unsuitable materials which had accumulated during the month-long period of exposure were removed from the foundation surface between stations 29+64 and 31+90.

4.61 CORE TRENCH - SURFACE TREATMENT. Surface treatment within the core zone consisted of the placement of 270 cubic yards of low-slump, 1000-lb/in^2 minimum compressive strength dental concrete containing 3/4-inch maximum size aggregate; and several cubic feet of grout slurry, mixed by hand in a ratio of one part sand to one part cement. The dental concrete was placed in cleaned and wetted large depressions and low areas to provide a more uniform surface for material placement and compaction, and to protect areas of highly fractured intact rock subject to possible damage from compaction equipment. A truck mounted concrete pump with an extendable boom arm (photo 53) was used to place the concrete, which was then consolidated by internal vibrating equipment, with special emphasis placed along the rock contacts to insure adequate bonding (photo 54). The surface was tamped where necessary to eliminate any potential planes of weakness and to provide a somewhat rough texture which would facilitate bonding with the core material. Only minimal concrete was required on the abutment slope and the concrete placed was contained by wooden forms. A curing compound was applied to the finished concrete surfaces to reduce cracking. Because of the extensive amount of dental concrete utilized (photo 55) and the presence of the green clay infilling material, only minimal amounts of grout slurry were placed. The use of grout slurry was restricted to the invert slope between stations 31+70 and 31+90 where residual soil rather than clay - infilling material was present. The slurry mixture was placed by hand in fully cleaned and prewetted fractures and voids between rock blocks which were inaccessible to compaction equipment (photo 56).

4.62 STAGE I FINAL CLEANUP. By 13 February 1984, the Stage I core trench was ready to receive embankment fill. Rock picks were used to chip off accidental concrete spills from the foundation surfaces and to trim loose and thin layers from the edges of dental concrete areas. The entire bottom was then reblown with a high pressure air hose and wetted prior to material placement. The final cleanup of the sloped portion of the core trench was done in 5 to 10-foot reaches, due to the slower rate of material placement against the steeper slope and consequently, the longer period of bedrock exposure.

4.63 ABUTMENT - STAGE III SURFACE PREPARATION. Foundation preparation and treatment of the right abutment core-transition contact zone after the completion of all grouting activities was accomplished during the Stage III construction period between October and December 1984. Following the same procedures established during foundation preparation of the Stage I core trench surface, the abutment surface was cleaned and treated in 8 sections, each an average 17 feet in length with an average elevation difference of 13 feet. This allowed the Contractor to work the abutment slope near the top of the embankment fill without having to use safety lines (photo 57). Each section was also within the placement reach of a crane hoisted concrete bucket. Much more extensive and detailed cleaning was required during this phase of foundation preparation before dental concrete, grout slurry, and embankment materials could be placed. Besides removing loose rock fragments and by-products of the drilling and grouting operations, numerous prominent open joints with apertures ranging from approximately 1/4 to 2-1/2 inches had to be cleaned to the specified depths to permit successful slurry grouting. In addition, the detailed cleaning out of high-angle, jumbled zones of shattered rock and residual soil between coherent intact rock blocks created localized voids and cavities up to 4 feet (photo 58) deep which were subsequently backfilled with dental concrete. Large, isolated protruding rock blocks were removed by pry bar if it was felt adequate compaction against them could not be achieved.

4.64 ABUTMENT - SURFACE TREATMENT/FINAL CLEANUP. Surface treatment within the core zone consisted of the placement of approximately 130 cubic yards of dental concrete using a concrete bucket and crane and the hand application of about 100 cubic feet of grout slurry (photos 59 through 61) Bulkheads were not constructed for concrete containment. Instead the water content of concrete placed in shallow depressions was varied to allow the concrete to stand on the steep slope and placement was done slowly and carefully to avoid excess spillage onto smooth intact rock surfaces. Shovels and small grout slurry "dams" were also frequently employed to buttress the concrete. Precautions were taken during vibration to insure that an adequate contact was developed between the rock-concrete interface and that the vibrating equipment did not cause the concrete to move down the slope. Grout slurry application was more extensive on the right abutment due to the greater number of open fractures infilled with unsuitable materials. Only low pressure air blasting was used during the final abutment cleanup to preclude disturbance of the treated foundation surface.

4.65 The one consistent and unavoidable problem encountered during foundation preparation and treatment in progressive stages up the abutment slope was the tendency for contamination of previously inspected and approved sections.

This required frequent re-cleanings of these areas if embankment placement did not keep pace with the abutment work. Due to the critical nature of Stage III construction, it was imperative that fill placement and abutment preparation operations occurring simultaneously in a limited work area be conducted in such a manner as to minimize delays. To prevent contamination of compacted core material near the base of the abutment, the Contractor placed a protective cover of loose core material on the ramped embankment surface. Once additional core material was ready to be placed, the loose fill and abutment debris was removed by backhoe.

Left Abutment/Core Trench

4.66 SURFACE PREPARATION. During excavation of the left abutment and Stage II core trench to station [illegible], a Case backhoe equipped with a scraper bar assisted the excavator in its initial cleaning effort (photo 62). Once the foundation rock was partially exposed, laborers with low pressure air hoses (photo 63) completed the preliminary cleanup by 13 January 1984, at which time they directed their efforts to the outlet works conduit excavation.

4.67 After foundation drilling and grouting of this portion of the Stage II excavation was completed, foundation preparation resumed on 28 February 1984 beginning at the top of the left abutment. Although actual foundation treatment of the left abutment and core trench in the vicinity of the outlet works was not done until several months later, detailed cleaning particularly along the dam centerline was required for geologic mapping purposes. Most of the residue from the drilling and grouting operations was removable by high pressure air blasting. However, laborers also cleaned the bedrock surface by hand and with shovels. The fairly uniform left abutment slope and the lack of any prominent open joints in the granitic rock made foundation preparation a rather quick and simple process. Two air cleanings of the partially completed Stage II excavation were accomplished in approximately one week. No further foundation cleanup was scheduled until just prior to surface treatment or embankment placement because of the anticipated long period of exposure and resultant continuing degradation of the bedrock surface.

4.68 Following completion of the remaining Stage II core trench excavation, the Contractor was permitted to initially clean off a narrow 40 foot wide strip along the dam centerline between stations 16+50 and 21+06 to expedite drilling and grouting operations. Foundation preparation of the remainder of the core zone, and transition zones was allowed to continue during drilling and grouting provided it did not interfere with the subcontractor's progress. Again, the preliminary clean-up was accomplished following the same sequence established previously. Backhoes with scraper bars, assisted by laborers with shovels, removed most of the unsuitable materials (photo 64), and subsequent high pressure air blasting exposed a foundation surface suitable for grouting and/or geologic mapping (photo 65). The muck generated from surface preparation was disposed using the same techniques described for the Stage I core trench cleanup. As the preparation of the core trench bottom progressed, it became apparent that normal cleaning methods would not be satisfactory for dislodging the cemented cobbles and boulders from the numerous bedrock scour channels. Laborers using shovels and pry bars and backhoes without scraper bars were necessary to perform this arduous and difficult task (photo 66). A

narrow single ripping tooth attached to the curved portion of the backhoe bucket was frequently used to clean out very narrow scour channels. Although vehicular traffic over an exposed bedrock surface was generally restricted to rubber-tired equipment only to protect the integrity of the foundation, concerns over the quality of the bedrock in localized areas between stations 16+00 and 16+90 required the use of a D9H dozer with slopeboard for corrective action. The dozer ripped several feet of the calichified granite cap near the upstream edge of the core trench excavation in the vicinity of station 16+40 (photo 67) and the slopeboard was used to flatten the steep bedrock slope to approximately 1V on 1H and remove soft highly weathered rock in areas of the slope not accessible to a backhoe or loader (photo 68). Additional excavation of the calichified granite cap outside the ripped areas was subsequently done with jackhammers. Surface preparation prior to foundation treatment consisted mostly of low pressure air blasting to remove any remaining loose rock and residue from the grouting program.

4.69 SURFACE TREATMENT/FINAL CLEANUP. Only dental concrete was required to satisfy foundation treatment requirements for the Stage II core zone. No grout slurry was necessary because of the tight or well healed nature of the bedrock discontinuities. A total of 538 cubic yards of dental concrete was used, mostly on the left abutment (photo 69). The concrete was placed in five reaches, the limits of which were dependent on the following interrelated factors: (1) equipment access, (2) placement reach of the hoisting crane, (3) fill placement schedule, and (4) foundation surface configuration. In the core trench, dental concrete needed to be placed in wider sections, due to the more rapid rate of embankment material placement over the relatively flat foundation surface. On the left abutment, however, the steeper slope grade, the limited reach of the crane, and the slower rate of fill placement necessitated dental concrete placement in much shorter sections, although not as restrictive as those on the right abutment. Foundation treatment was necessary to backfill large depressions in the bedrock surface, to protect fractured, weathered bedrock from possible degradation during initial fill placement and compaction, and to facilitate equipment placement and compaction over the originally locally irregular foundation surface. Between the outlet works and station 21+06, dental concrete was used mainly to backfill the extensive network of scour channels where adequate compaction would have been difficult to obtain, thus leading to the possible development of seepage paths through the embankment core (photos 70 and 71). The preconstruction test trench downstream of the dam centerline between stations 16+00 and 16+50 also required backfilling because of its high vertical sidewalls (photo 72). A sloping bulkhead was constructed at the open west end of the trench to contain the concrete. Approved surface treatment procedures were again followed prior to and during dental concrete placement. Each designated area was initially cleaned with a low pressure air hose and then wetted. After the concrete was placed, either by bucket or when possible, directly from the truck chute, it was vibrated, and the surface tamped. Prior to fill placement, the edges of all concrete areas were trimmed of all loose or thin layers and the foundation surface given one final blow down.

Foundation Drilling and Grouting

4.70 GENERAL. As part of the dam foundation treatment, subsurface pressure grouting was conducted along those portions of the core zone founded on bedrock. The intent of the grouting program was twofold: to serve as an extension of the design investigation to assure that no pervious zones or voids crossed the foundation, and to improve the foundation by reducing the permeability of the rock mass to preclude piping of embankment materials through open joints and fractures. Drilling and grouting operations commenced on 10 January 1984 and were completed in a satisfactory and timely manner on 21 March 1984. Supervision and inspection of these operations were provided by Corps of Engineers geologists. A total of 216 holes were drilled and grouted along or near the dam centerline, forming a single line curtain ranging in depth from 25 to 75 feet. The grout curtain extended from the top of the left abutment across the bedrock portion of the core trench to the top of the right abutment. The final cost of the grouting program was approximately $170,000.

4.71 CONTRACT SPECIFICATIONS. The specifications for the foundation drilling and grouting program were written utilizing information from the following sources: (a) guide specification CE 1305.01, dated October 1959, (b) drilling and grouting specifications from previous District construction projects, and (c) available literature on grouting methods and equipment. Payment items, estimated quantities and actual contract prices were as follows:

Mobilization/demobilization	1 Job	$20,000
Drilling exploratory grout holes	350 LF	$ 25/LF
Drilling grout holes	9,100 LF	$ 8/LF
Pipe for grout holes	500 LF	$ 5/LF
Drill set-ups		
(1) Grout holes	370 EA	$ 25 EA
(2) Exploratory grout holes	4 EA	$ 50 EA
Pressure testing	175 HR	$ 60/HR
Grout pump connections	375 EA	$ 50 EA
Placing grout	2,500 SACKS	$ 20/SACK

4.72 The estimated quantities were determined from the proposed grouting scheme shown on plate 45. It was decided a single-row, maximum 3-zone grout curtain with a 10-foot primary hole spacing would be adequate for a flood-control dam without a positive cutoff. Each zone would be 25 feet in length and the maximum number of zones required between certain elevations was determined using 2/3 the height of the dam embankment core zone as a practical guide. Therefore, grout hole depths would be 25 feet between elevations 1482 and 1440, 50 feet between elevations 1440 and 1400, and a maximum of 75 feet below elevation 1400. In addition, it was anticipated that approximately 20 percent of the grout holes would require split-spacing and that grout takes would average approximately 0.3 sacks per linear foot of hole. It was assumed that the bedrock foundation, despite the fractured nature and variable permeability of the rock mass, was relatively impermeable and would not accept large quantities of grout due to the typical narrow separation between joint surfaces and the widespread presence of infilling material along joint planes.

4.73 PREMOBILIZATION. The Contractor selected the W.G. Jaques Company of Des Moines, Iowa to be the drilling and grouting subcontractor. Since the Jaques Company had also performed the foundation grouting for the Adobe Dam project and was already familiar with similar contract specifications and general operating and reporting procedures, no major problems were anticipated or encountered prior to the start of mobilization. Correspondence between the Corps of Engineers and the Jaques Company was initiated several months in advance of the expected mobilization in order to ensure a timely start and efficient operation of the grouting program and to avoid impacting the Contractor's accelerated construction schedule. Information regarding the subcontractor's proposed drilling and grouting equipment, work schedules, wastewater control plan, and safety program was requested. In their submittal, Jaques anticipated using a combination of percussion and rotary drills to accomplish the work. In addition, the grout plants could be equipped with high-speed mixers to more effectively mix and stir the grout. Jaques proposed an open-system wastewater control plan, stating the closed-system collection and disposal set-up (similar to the one used at Adobe Dam) was neither practical, effective nor necessary in preventing defacement or damage to the cleaned foundation surface. Under the open-system plan, wastewater would be allowed to flow down the natural bedrock slope to a point where it could be collected in a temporary sump. The wastewater could then be pumped to a suitable disposal site. Any accumulation of large quantities of solids in the sump could also be removed to a designated disposal area. The submitted wastewater control plan was approved with the stipulation that all precautions had to be taken to preclude any degradation of foundation surfaces or contamination of embankment fills, and that any impact upon construction activities due to wastewater or grout within the construction area would be at the expense of the Contractor.

4.74 MOBILIZATION. Although a preliminary construction schedule had shown drilling and grouting operations possibly beginning as early as mid-November 1983, the actual mobilization of Jaques Company personnel and equipment to the site did not occur until 3 January 1984. The change in schedule was due in part to delays in the west abutment and Stage I core trench excavations. Jaques Company personnel consisted of a project manager, a project superintendent, an assistant project superintendent, and a maximum of 7 local laborers/drillers. The equipment consisted of 2 air track percussion drill rigs, one Longyear 24 rotary drill rig, 2 grout plants, and all required appurtenances. Before any work actually commenced, a formal mutual understanding meeting, attended by representatives of the project engineers staff, the Geology Section, Sundt, and Jaques was held on 4 January 1984 to discuss drilling and grouting work schedules and procedures, the wastewater control plan, general safety requirements, applicable specifications, and payment item recording procedures. In addition, an on-site hazard analysis meeting was held on 10 January 1984 just prior to commencement of work to thoroughly review all applicable safety requirements and procedures, particularly when working on the steep, irregular west abutment surface.

4.75 DRILLING AND GROUTING EQUIPMENT. The drilling and grouting equipment used to perform the work conformed to the contract specifications except for those variations approved by the Project Geologist. The equipment used can be summarized as follows:

a. Drills for grout holes. The subcontractor elected to use Ingersoll-Rand air track percussion drills equipped with 2-1/2-inch diameter diamond plug bits to drill all the grout holes (photo 73). An air-water mixture was used as a circulating medium to remove drill cuttings from the grout holes. This equipment satisfied the contract specification requirements.

b. Drill for exploratory grout holes. A Longyear 24-Standard skid-mounted rotary core drill equipped with an NW double tube core barrel was used to drill all the exploratory grout holes. Although the drill rig lacked some features which would have expedited the exploratory coring, the rig and accessory equipment were more than adequate to complete the work.

c. Grout Plants. Two Jaques over/under portable grout plants Model GP-16 were approved for use during all grouting operations. Each plant was equipped with at least one 16 cubic foot mixing tub (although one plant had 2 mixing tubs), an 18 cubic foot agitator (sump), and a Moyno 3 stage helical screw progressive cavity pump. Air motors rated at 3 horse-power were used for mechanical paddle mixing of the grout. Colloidal mixing, utilizing an additional 9 horse-power air motor powering a minimum 1500 rpm centrifugal pump, was also attempted for a short time during the initial stage of grouting. This method proved ineffective when grouting "high-take" holes since the longer two-stage mixing process involving both paddle and colloidal mixing severely reduced the capability of the grout plant to deliver a continuous supply of grout to the hole. The paddle-type mixer proved capable of effectively mixing and stirring grout with various water-cement ratios.

d. Grout header. A direct grouting header (see fig. 8), essentially the same as that specified on plate 45, was used during grouting (and pressure testing) except that the header was connected to a downhole supply line and not the grout pipe. The circuit grouting header specified was not required.

e. Accessory equipment. All accessory equipment and supplies, including pressure gages, water meters, and cement conformed to the requirements of the contract specifications.

4.76 DRILLING AND GROUTING PROCEDURES. Foundation drilling and grouting was accomplished using the specified stage grouting method, which involved the drilling and subsequent grouting of progressively deeper zones in stages. Stage grouting procedures conformed to the contract specifications except where variations were required or permitted. The typical procedures used are described in the following paragraphs.

a. After excavation in a particular area had been completed, the full widths of the core and transition zones were generally thoroughly cleaned to allow concurrent grouting and geologic mapping activities. However, in some instances, cleaning was concentrated in a 20 to 40-foot wide zone along the dam centerline to expedite the grouting program.

b. Prior to any grout hole drilling, the core zone centerline was established by survey. Stations were marked on 5-foot centers on each abutment and on 10-foot centers in the core trench, with elevations recorded for each station.

c. Primary grout hole locations were marked as planned with minor adjustments as necessary to fit local geologic or topographic conditions.

d. First stage primary grout holes were drilled with 2-1/2-inch diameter diamond plug bits to the depth of the specified zone, which was usually 25 feet but occasionally 15 feet. The entire stage grouting process within a given zone could be done in one operation because zones of complete circulation loss were difficult to delineate using the percussion drilling method.

e. After drilling was completed, the drill bit was raised off the bottom of each grout hole and the hole was pressure washed using an air-water mixture. Pressure washing was continued until the return water was clean or for a maximum period of 10 minutes. Before the drill rods were removed, air was injected into the hole to force most of the water out and to prevent drill cuttings from re-entering the hole.

f. The completed grout hole was then capped using a wooden plug wrapped in a burlap sack to prevent contamination. Installation of grout pipe was not reguired for the following reasons: (1) the packer grouting technique used allowed the grout header to be attached to the downhole supply line instead of directly to a grout pipe; (2) grout pipe was not necessary to maintain hole alignment during percussion drilling, or to reduce surface leaks during packer grouting; and (3) the rock surface was competent enough to provide a firm hole collar and withstand any severe hole washout due to the erosive action of the circulating medium.

g. Each hole was pressure tested with water normally for a 5 to 10 minute period or until a uniform flow rate was established at the gage pressure specified, or until surface leaks developed. Generally only the maximum gage pressure (expressed in pounds per square inch) was used for each test interval. For zone I pressure testing, a vertical depth of 10 feet was normally used in the calculation of maximum gage pressure. For Zones II and III, the vertical depth to the top of the respective zone was used for all calculations.

h. If the permeability (K value) calculated from pressure test data for a given zone was less than 0.1 feet per day, grouting of that portion of the hole was deferred. Experience had shown that grout with a water-cement ratio of 6:1 could not be injected into holes meeting this criteria and would, in essence, just fill up the hole. The method to obtain permeability values is explained in the Foundation Grouting Summary notes in table 8.

i. If pressure test data showed a calculated permeability of 0.1 feet per day or greater, grouting was begun using a thin mix of 6:1, regardless of the suspected eventual take. Grout mixes were thickened incrementally, as required, to preclude premature stoppage. All grout mixes consisted of water and Type II cement; no additives or sand were used. Grouting of clean holes was accomplished using the packer grouting method in lieu of the direct grouting method. Under this arrangement, the direct grouting header was connected to a downhole grout supply line with a pneumatic packer attachment capable of being set in the hole at the depth specified. For Zone I grouting, a packer setting near the top of the hole (normally at a depth of 2 feet

unless hole washout prevented the packer from seating properly) was used. For Zone II and Zone III grouting, shallow packer settings were also used since it was assumed that grout injection would occur in the zone being grouted and not in any previously grouted or deferred zones. The circuit grouting header suggested for use in caving holes was not required because hole caving was never a significant problem.

j. Surface leaks were rarely a problem during grouting. Occurrences were generally limited to the west abutment where wider, cleaner joints were more prevalent. Leaks were commonly calked with oakum or quick setting mortar.

k. The maximum gage pressure used in grouting a particular zone was usually the same as the maximum gage pressure used during water testing. However, gage pressures were occasionally adjusted when grouting with thicker mixes to account for the increased grout column pressures.

l. Generally, once grouting of a hole had begun, it was continued to completion without any interruption. However, in two instances, grouting was suspended until the following work day; once because of equipment breakdown, and once due to the unavailability of a light plant for night work. In several other cases, grout was allowed to circulate in the lines for short periods of time to allow calking of surface leaks or because of temporary conflicts with the prime-contractor's work schedule.

m. The grouting of any hole was not considered completed until that hole refused to take any grout whatsoever at 3/4 the maximum gage pressure required for the stage or when that hole took grout at the rate of 1 cubic foot or less in 10 minutes measured over at least a 5-minute period at the maximum gage pressure required for the stage.

n. Once grouting was completed, the hole was recapped and the grout was required to set for a minimum period of 16 hours before the hole could be deepened or drilling within 50 feet of the grouted hole could commence. In lieu of washing out grout preparatory to drilling deeper stages, the subcontractor elected to redrill at his own expense.

o. Split-spaced grout holes (secondary through quinary) were generally not required unless grout takes exceeded 0.3 sacks per linear foot of hole or 7-1/2 sacks per 25 foot zone. Occasionally split-spacing was done, not because of high grout takes, but to check the effectiveness of the grouting program.

4.77 GROUTING SEQUENCE. The sequence of drilling and grouting specified for sections in a grout area was generally adhered to except when construction schedules or field conditions dictated otherwise. However, grout sections as defined in the contract specifications were never formally established. Instead, time and distance requirements outlined in other portions of the specifications were used to establish when and where drilling and grouting could be performed. That is, no drilling was permitted within 50 feet of a grouted hole regardless of the zone grouted until at least 16 hours had elapsed since grouting. In addition, all stage grouting work in a given zone and reach had to be completed before work could start in the next underlying

zone. Grouting was usually begun at the lower elevation of a reach, particularly on the abutments, to facilitate construction of a more effective grout curtain.

4.78 GROUTING LIMITS. A single-row grout curtain was formed along the dam centerline from station 10+10 on the left abutment to station 21+01 in the Stage II core trench and from station 29+69 in the Stage I core trench to station 33+17 on the right abutment. Between stations 21+06 and 29+64, the dam embankment was founded on alluvium so grouting was not required. For purposes of discussion, the foundation grouting program has been divided into two sections; the west side, which includes the west (right) abutment (stations 31+90 to 33+27) and Stage I core trench (stations 29+64 to 31+90); and the east side, which includes the east (left) abutment (stations 10+00 to 13+00) and Stage II core trench (stations 13+00 to 21+06). Foundation conditions were different for each section. The west side grouting was conducted in mostly hard, highly fractured, unweathered andesite bedrock while the east side grouting was conducted in typically highly fractured and sheared granitic bedrock with variable hardness and weathering characteristics. The grouting program also included the drilling of 5 exploratory grout (core) holes to check the effectiveness of the grouting and to obtain additional information on subsurface conditions. Data for each grout hole and exploratory grout hole are summarized in table 8 and are graphically shown on plates 34 through 36. A comparison of average grout takes within zones of each section is shown in table 7.

4.79 PERCUSSION DRILLING. Despite Government reservations over the use of tracked drilling equipment on exposed bedrock surfaces, the Contractor was able to carefully and effectively maneuver the drills over the sometimes irregular foundation surfaces without severely damaging or degrading the volcanic and granitic bedrock. On the right abutment, the slope was too steep for the drill to operate safely and effectively without assistance from a winch cable system (photo 74). Two pairs of 2-inch steel "tee" bars, set approximately 5 feet into bedrock, were installed as anchors for the winch cable arrangement. One pair of tee bars was located on the abutment staging platform to provide anchorage on the upper half of the abutment and the other pair was located at approximately mid-slope to service the lower half of the abutment. The arrangement of the winch cable was such that the winching of the drill into place could be accomplished utilizing a primary cable supplemented by a safety cable tie off. Because of the potential slope hazards, certain steps were taken to insure a greater degree of safety during drilling and grouting operations. They included:

a. All personnel working on the slopes were required to use safety lines.

b. When moving between grout holes, the main section (mast) of the drill was set in its lowest position for increased equipment stability.

c. During angle hole drilling, drill steel, cut in shorter 6-foot lengths, was used for easier handling and to improve the stability of the drill.

d. The subcontractor was allowed to use smaller than specified 1-1/4-inch circulation lines for easier handling on the steep slope.

Despite the potential problems involved in working on a steep, irregular slope, Jaques completed the required work in a timely and efficient manner.

4.80 WEST SIDE GROUTING. Drilling and grouting operations on the west side were conducted at two different times to avoid impacting the Contractor's construction schedule and field operations. The core trench and lower portion of the west abutment between stations 29+69 and 32+17.5 (hereinafter referred to as Stage I grouting) was grouted between 10 January and 8 February 1984, just prior to Stage I fill placement. The remaining portion of the west abutment between stations 32+17.5 and 33+17 (hereinafter referred to as Stage III grouting) was grouted between 29 February and 8 March 1984, after completion of most Stage I construction activities.

4.81 Stage I grouting began after the preliminary cleanup of the abutment and core trench surfaces had been completed. Work progressed from the base of the core trench up the abutment slope to approximate elevation 1406. This elevation, 13 feet above the top elevation of the Stage I fill and protective cover, and 5 feet above the top elevation of the Stage II diversion levee, was established as the upper limit of the initial grouting phase for the following reasons: (1) the 13-foot buffer would provide sufficient overlap should additional split-spacing be required during Stage III grouting, (2) no additional Zone III grout holes would need to be drilled, and (3) any degradation or contamination of the grouted abutment surface from subsequent flood flows through the closure gap would not impact or delay Stage III grouting activities.

4.82 Stage III grouting began after construction of the Stage I embankment had reached elevation 1380 and the core zone covered by a 2-foot protective cap of erosion resistant material to preclude contamination of the underlying fill. Work resumed on the west abutment earlier than anticipated and prior to completion of Stage I activities because delays in completing the Stage II core trench excavation had caused Jaques to temporarily suspend operations on the east side. All drilling and grouting was completed before the Stage I diversion levee, which afforded protection to the Stage I work area, was completely removed.

4.83 Primary grout holes on the west side were drilled on approximately 10-foot centers between stations 29+69 and 31+70. Between stations 31+70 and 33+17, the horizontally measured hole spacing was reduced from 10 feet to an average of 7 feet to compensate for the steepness of the abutment slope. The holes were located at or near the dam centerline depending on surface conditions. A grout hole inclination of 30 degrees, measured from the vertical and oriented toward the slope, parallel to the dam axis, was selected to maximize intersection of the dominant joint patterns in the andesite bedrock. However, based on a recommendation by SPD Geotechnical Branch, this standard arrangement was changed for holes 29+70P and 29+80P to create a "fan" pattern for the purposes of intercepting and grouting the sloping bedrock under the alluvial portion of the core trench. Grout hole 29+70P was drilled at an inclination of 25 degrees to the east while hole 29+80P was drilled vertically.

4.84 Forty-one primary grout holes and 7 secondary grout holes were drilled to the full 25 foot depth of the first zone. A total of 96.7 sacks of grout with water-cement ratios of 6:1 to 2:1 were injected with an average grout take of 0.08 sacks per linear foot of hole (see table 7). Calculated rock mass permeabilities ranged from 0.0 to 2.8 feet per day with an average grout hole permeability of 0.5 feet per day. Due to the pervasiveness (both laterally and vertically) of the highly plastic, non-dispersive green clay fracture filling in the core trench, grouting was deferred in 18 of 23 primary holes drilled between stations 29+64 and 31+90. In fact, a Zone I permeability of 0 feet per day was calculated for the reach between stations 29+70 and 31+30 compared to an average rock mass permeability of 0.1 feet per day for the core trench as a whole. An average permeability of 1.0 feet per day was calculated for the more permeable Zone I abutment rock, which was characterized by open, residual soil filled fractures and a greater susceptibility to surface leaks during pressure testing and grouting. The abutment also accepted a larger quantity of grout (94 sacks total), with takes averaging close to 0.2 sacks per linear foot of hole. In comparison, only 2.7 sacks could be injected into 5 core trench holes, resulting in an average grout take of less than 0.01 sacks per linear foot. Grout takes on the abutment were not uniform, however, with two small reaches, between stations 32+05 and 32+17 and between stations 32+30 and 32+37, accounting for nearly 90 percent of the total grout placed. In fact, nearly half of the total (40.7 sacks) was injected into hole 32+10P alone. Results of the grouting appear to indicate that these high take areas respresent localized void or cavity filling and not an extensive interconnected open fracture condition. Grout injection rates tended to decline very sharply prior to absolute refusal and grout takes in holes adjacent to the high take areas were nominal. Five secondary holes were required to bracket primary holes 32+10P, 32+17P and 32+30P in order to establish a more effective grout curtain to the first zone depth. Two secondary holes, 32+79S and 32+86S, were drilled to explore the unusually tight foundation conditions of the upper abutment and to verify the effectiveness of the grouting program.

4.85 Once the Zone I grouting in any specified area had been completed according to the criteria established in paragraph 4.76, Zone II grouting was initiated. All first zone primary grout holes and one secondary grout hole between stations 29+70 and 32+60 were deepened to the full 50-foot depth of the second zone. Required Zone II secondary and tertiary holes were drilled and grouted to full depth in one operation, not in stages, because it was assumed the upper zone had already been satisfactorily grouted. Results obtained from grouting indicated a slightly less permeable intermediate zone, with an average calculated permeability of 0.3 feet/day. The remarkable consistency of the individual rock mass permeabilities, with a much narrower range from 0.0 to 1.5 feet/day, contributed to the overall lower Zone II average. Grout takes, however, were 2-1/2 times higher for Zone II. Grouting was deferred for 11 holes (including all 4 tertiary holes) and 239.2 sacks of 6:1 to 1:1 grout were placed in the remaining 33 holes. The average take in the grouted holes was 0.2 sacks per linear foot, including 0.3 sacks/ft. for abutment holes and 0.2 sacks/ft. for core trench holes. Again, these figures were not a true reflection of Zone II grouting as a whole. Over 60 percent of the grout was placed in just 3 holes, 29+70P, 29+75S and 32+10P. The first 2 holes were part of the fan pattern drilled near the bedrock margin of the core

trench and the possibility of grout traveling out into the more pervious alluvium may account for the abnormally high grout takes (76.4 sacks total). On the abutment, the highest grout take again occurred in hole 32+10P. A total of 77.2 sacks, or 86 percent of the abutment total, were placed; an amount about 30 times greater than that injected into any nearby Zone II primary grout hole. Grouting was deferred for Zone II of secondary grout hole 32+05S and further split-spacing was considered unnecessary due to the tight hole spacing on the abutment. Tertiary holes were drilled between stations 29+70 and 29+90, not only to bracket high-take secondary holes but to explore those areas between adjacent holes, which, due to varying hole inclinations necessary to create the fan pattern, exceeded the specified primary or split-spaced hole spacing at depth.

4.86 For Zone III grouting, a primary hole spacing based on 20-foot centers was selected as a basis for design. In other words, alternate primary holes would be drilled to 75 feet with the remaining primary holes considered secondary holes and deepened as required. This was done to reduce the amount of drilling footage by utilizing existing primary holes as secondary holes, and to bring the grouting program costs more in line with the design intent of the dam and existing foundation conditions. The third zone proved to be the most permeable of the three zone grout curtain. An average rock mass permeability of 0.4 ft/day was calculated for this 25 foot section, and 607.8 sacks of 6:1 to 1:1 grout were injected into 33 grout holes. This figure represents an average take of 0.7 sacks per linear foot, or approximately 3-1/2 times the amount of grout placed in the overlying Zone II rock. For purposes of analysis, the following primary grout holes: 30+10P, 30+30P, 30+50P, 30+70P, 30+90P, 31+10P, 31+30P, 31+50P, 31+70P, 31+90P, and 32+00P, were considered secondary holes due to the 20 foot primary hole spacing established for Zone III. As a consequence, this necessitated the following secondary holes: 30+15S, 30+65S, and 30+75S being considered tertiary holes in the analysis of data. This was done in an attempt to normalize the grouting data for comparison between hole series in Zone III or different zones as shown in table 7. The wider primary hole spacing indicated that although initial grout takes were high (averaging 37 sacks per hole), an effective grout curtain could be constructed by split-spacing on 10-foot centers. In effect, all original primary holes were deepened to the full 75 foot depth and only the 3 "tertiary" holes were needed to seal off more pervious zones. Most of the Zone III grouting was accomplished in the core trench with the right abutment accounting for less than 10 percent of the total. Unlike the previous zones grouted, high grout takes for primary holes were much more common and evenly distributed. Seven holes took in excess of 50 sacks each while overall, average grout takes of 1.6 sacks per linear foot of hole and 1.0 sacks per linear foot of hole were calculated for core trench and abutment holes, respectively. However, grout takes declined drastically for split-spaced secondary through quaternary holes, indicating that the network of open, cleaner fractures at depth was apparently localized and did not display a high degree of lateral continuity. To tighten up the fan pattern at depth between stations 29+70 and 29+90, 2 tertiary holes (29+82.5T and 29+87.5T) and 1 quaternary hole (29+78.3Q_1) were drilled; however, grouting of each hole was deferred. The apparent increase in average grout take between Zone III secondary and tertiary holes is very misleading. Only one grout hole, 29+78.5T, required grouting and the reason was its close proximity to the bedrock/alluvium contact.

4.87 Once all the required grouting was completed, two vertical NW-core holes; 100C (station 31+28.5, 6 ft. upstream of centerline) and 101C (station 29+79, 5.5 ft. upstream of centerline), were drilled to check the effectiveness of the west side grouting program and to further explore the subsurface geologic structure. See figures 3 and 4 for the logs of exploratory core holes 100C and 101C, respectively. Both holes were drilled using a Longyear-24 skid rig to depths of approximately 66 feet without any circulation losses. Green clay infilling material was generally present throughout the highly fractured rock cores with the heaviest concentration occurring in the upper 3 to 7 feet of Zone I, where rock mass permeabilities and grout takes were the lowest. A composite RQD (Rock Quality Designation) of 25 percent attests to the highly fractured nature of the andesite bedrock. Grout was not present along any joints or fractures above 48 feet in hole 101C and above 61 feet in hole 100C. Even when present, the grout did not necessarily completely fill a particular fracture; occasionally it was mixed with the clayey infilling material. Both holes accepted small quantities of grout (see table 8) but not amounts significant enough to require further exploration.

4.88 EAST SIDE GROUTING. Prior to starting grouting operations on the east side, the decision was made to delete all Zone III grouting with the exception of 2 exploratory holes. With the concurrence of SPD Geotechnical Branch, this action was taken for the following reasons:

a. The primary purpose of New River Dam is flood control; no recreation or conservation pool is planned.

b. The criteria used to determine grout hole depths are quite conservative. A more realistic approach would be to use two-thirds the maximum reservoir height to the spillway crest elevation of 1456 to figure the maximum grout curtain depth. This would result in a two zone curtain with a bottom vertical depth of approximately 44 feet.

c. The standard project flood (SPF) up to spillway crest elevation has an estimated 400-year frequency of occurrence.

d. The dam embankment is not founded on bedrock across the entire width of the valley. The central portion is founded on more permeable alluvium with a calculated average field permeability of 8 ft/day.

e. Given the conservative design of the embankment and the planned foundation treatment combined with the short duration flood pool, a maximum grout curtain depth of 50 feet was considered more than adequate in providing underseepage protection.

4.89 Drilling and grouting operations on the east side were conducted in two phases because of the Contractor's construction schedule. The left abutment and core trench from station 10+00 to station 16+05 was completely grouted between 17 January and 24 February 1984, after the outlet works and initial Stage II excavation had been completed. The remainder of the Stage II core trench between stations 16+05 and 21+06 was grouted between 13 March and 21 March 1984, once the Stage I construction was completed and the Contractor concentrated his efforts on completing the Stage II excavation.

4.90 The same drilling and grouting methodology employed on the west side was followed on the east side. Primary grout holes were established on 10-foot centers along the dam centerline or as close as permitted by the bedrock relief. A grout hole inclination of 30 degrees, measured from the vertical and oriented toward the abutment slope, was again selected to maximize intersection of the dominant joint patterns in the granitic bedrock. The drilling of grout holes in a fan pattern at the edge of the grout curtain to intercept the sloping bedrock surface under the alluvial portion of the core trench, as previously recommended by SPD Geotechnical Branch, was done at station 21+00.

4.91 Despite the more extensive drilling and grouting program required for the Stage II excavation, the work was accomplished in approximately the same amount of time as the west side grouting. To maintain this consistency in scheduling while at the same time doubling his efforts, the subcontractor utilized up to two air-track drills and grout plants where practical. In general, when grouting could be done concurrently in widely spaced sections, particularly on either side of the outlet works conduit, two distinct and separate operations were normally conducted because of the 200-foot length restriction placed on grout delivery lines (from sump to header) by the contract specifications, and problems with personnel and equipment access.

4.92 During Zone I grouting, a total of 109 primary grout holes were drilled to the full depth of the first zone. In all but 3 instances, the specified depth was 25 feet. To accomplish the near surface grouting of the dam foundation underlying the outlet conduit before actual construction commenced, holes 14+20P, 14+30P and 14+[illegible]0P were drilled vertically to depths of 15, 30, and 15 feet, respectively. Completion of the grout curtain at depth under the conduit structure and vicinity required a combination of vertical grout holes, and inclined grout holes with a westerly bearing because of the 40-foot interruption in the standard 30°E pattern. Grouting was deferred in 64 holes and only 12 split-spaced holes were required, with 8 of them located in the reach between stations 16+05 and 16+35. A total of 208.5 sacks of grout with water-cement ratios ranging from 6:1 to 2:1 (but rarely thicker than 6:1) were placed during Zone I grouting, resulting in an average grout take of 0.07 sacks per linear foot of hole. However, this average grout take figure is not representative of the first zone as a whole since approximately 60 percent of the total grout was injected into only 5 holes (13+50P, 16+20P, 16+22.5T, 16+25S, and 16+30P). Grouting was deferred in the entire reach between stations 18+70 and 21+06 and only 5.7 sacks were placed west of station 16+50. Calculated rock mass permeabilities for Zone I ranged from 0.0 to 2.9 feet per day with a weighted average of 0.4 feet per day. Exempting the 5 high-take grout holes, the average rock mass permeability declined to 0.3 feet per day. Overall, differences in grout take between the abutment and core trench were relatively minor; both areas accepted quantities of grout averaging less than 0.1 sacks per foot. Only two localized permeable areas were encountered in the upper 25 feet of the granitic bedrock and both were located in the core trench. Two secondary holes were necessary to bracket grout hole 13+50P which took 31.3 sacks of grout, while fourth order holes were required to effect closure of the previously mentioned high take area centered around station 16+22. What is interesting about this last area was the unexpected increase in grout takes from primary through tertiary holes in

a narrow 5-foot section between stations 16+20 and 16+25. Primary grout hole 16+20P required 17.3 sacks to complete, while secondary hole 16+25S and tertiary hole 16+22.5T accepted 30.1 and 33.2 sacks, respectively, at even thicker mix ratios before refusal. Only when quaternary holes 16+21.3Q_1 and 16+23.8Q_1 were drilled was this area successfully grouted. The reason for this phenomenon is unknown but either extremely localized subsurface conditions or undetected premature stoppage during grouting may be potential causes.

4.93 Following the completion of all necessary Zone I grouting in a particular area, Zone II grouting was initiated. This involved the deepening of all primary holes between stations 11+00 and 21+01, with the exception of outlet conduit holes 14+20P through 14+40P, to a depth of 50 feet. Required split-spaced secondary through quinary holes were again drilled and grouted to full depth in one operation based on the assumption that the upper zone had already been satisfactorily grouted. Analysis of the grouting data indicated that, like on the west side, the Zone II bedrock was, on the whole, more permeable. Although the same average rock mass permeability of 0.4 feet per day was calculated, a larger quantity of grout was injected into this zone, almost three times the amount placed in Zone I. As a result, this necessitated a greater degree of split-spacing to satisfactorily construct the grout curtain. Grouting was deferred for 51 primary holes and 1 secondary hole, and 493.8 sacks of 6:1 to 1:1 grout were placed in the remaining 49 primary and 27 split-spaced holes. The average take in the grouted holes was 0.2 sacks per linear foot. The higher grout takes probably reflect more competent, less weathered bedrock with cleaner, more open fractures. The same non-uniform pattern of grout injection characteristic of the Zone I holes was also evident for Zone II. However, the high take areas encountered were restricted to between stations 12+80 and 14+00, east of the outlet works and below elevation 1370. An intensive split-spacing effort, requiring fifth order or quinary holes was necessary to effectively grout two of these localized areas (see detail "A" on plate 36). Similar subsurface conditions were encountered during preconstruction geotechnical investigations near the base of the left abutment. Noticeable water losses during drilling of core holes DD-16, DD-19, DD-34, and DD-35, began to occur below approximately elevation 1380. This possible interconnected, permeable zone probably corresponds to the high take area intercepted during the lower abutment grouting. The two major high take areas were not very extensive. The first area, between stations 12+80 and 13+10 near the abutment-core trench interface, was grouted using a total of 196.6 sacks of 6:1 to 1:1 grout in 15 holes.

4.94 Despite an unexplained deviation from the theoretical pattern of decreased average grout takes during incremental split-spacing, it was assumed a satisfactory grout curtain had been placed prior to the drilling of exploratory core hole 102C. Split-spaced quinary grout holes (12+90.6Q_2 and 12+91.9Q_2), on approximate 0.5-foot centers, were necessary to bracket abnormally high-take quaternary hole 12+91.3Q_1 which took more grout (43.1 sacks) than any other grout except primary hole 13+00P. However, despite the close hole spacing, a complete loss of circulation occurred at 38 feet (elevation, 1359 foot) in hole 102C (see fig. 5) and 57.1 sacks of 6:1 to 1:1 grout were required to establish refusal. Examination of the granite rock core revealed a general lack of grout filled fractures below a depth of 34

feet, and several highly fractured to shattered zones with scattered calcite lined open joints between the depths of 38 and 40 feet. The apparent absence of grout in the core near the bottom of the grout curtain may indicate the intersection of a more permeable zone extending below the effective depth of the grout curtain, or the lack of complete grout penetration.

4.95 Since it now appeared further grouting was required, the decision was made to bracket the entire high take area with tertiary holes 12+87.5T and 13+02.5T. This action was taken because the close grout hole spacings made further split-spacing almost impossible and an examination of the grouting profile indicated that the water-loss zone in hole 102C apparently was near hole 13+00P. Although, hole 13+02.5T could have been split-spaced, according to the criteria set forth in paragraph 4.76, grouting was discontinued for the following reasons: (1) an adequate investigatory program had been conducted; and (2) in the case of those Zone II split-spaced grout holes which could be drilled to full depth in one operation without being stage grouted, the 0.3 sack per linear foot limit was probably a little too rigid for such holes. For the sake of simplicity, it was assumed that grout injection occurred only within the lowermost zone being grouted and that the upper zone would not take grout because adjacent holes had shown the area to be relatively tight.

4.96 The other major localized zone of high grout take was situated between stations 13+35 and 13+55. In this reach, 180.2 sacks of 6:1 to 1-1/2:1 grout were placed in 14 holes. An almost identical situation developed during the exploration of this reach. Average grout takes increased progressively for primary through tertiary holes and then dropped abruptly for the quaternary holes. Again, despite this abnormal trend and the fact that one quaternary hole (13+46.3Q_1) exceeded slightly the recommended minimum split-spacing requirement of 7.5 sacks per zone, it was felt the grouting program had been successfully completed. However, a 100 percent loss of circulation also occurred in exploratory core hole 103C, at a depth (36 feet) and corresponding elevation (1357 feet) consistent with that in hole 102C (see fig. 6). Drilling was stopped at 38 feet and the hole stage grouted. A total of 73.2 sacks of 6:1 to 2:1 grout, a quantity far exceeding that placed in any one grout hole, were placed. The core hole was subsequently deepened to a depth of 45 feet to further check the effectiveness of the grouting program. The results were positive; no circulation losses were noted and pressure test data indicated a tight zone. Unlike hole 102C, grout lined fractures were present in the granite rock core to the bottom depth of hole 103C. However, it appeared that the complete loss of drill water circulation occurred either in a rubbly zone with several open voids or along open, apparently ungrouted fractures. Since further grouting was indicated because of the subsurface conditions encountered, quinary holes 13+45.6Q_2 and 13+49.6Q_2 were drilled to bracket quaternary hole 13+46.3Q_1, which took 8.3 sacks of grout. Even with hole spacings as close as 0.5 feet, it was still possible to inject a total of 13.2 sacks into the foundation rock, including 9.0 sacks in hole 13+46.9Q_2 alone. However, further grouting was also deemed unnecessary for the reasons stated in the preceding paragraph.

4.97 Grouting of the remainder of Zone II on the east side was fairly routine. One other small high take area between stations 13+80 and 14+00 was successfully grouted using just secondary holes. West of the outlet works the

foundation was remarkably tight. Although secondary hole 16+35S was deepened to check on the possible deficient grouting of adjacent primary holes 16+30P and 16+40P, no split-spacing was actually required. Of the 63 primary holes drilled, grouting was deferred in 44 holes and only 27 sacks of thin grout were placed in the remaining 19 holes. The average grout take for this 650 foot section was an extremely low 0.02 sacks per linear foot.

4.98 Two Zone III "exploratory" grout holes were drilled for the purpose of checking the foundation conditions below the established grout curtain elevation of approximately 1340 feet. The rock mass permeabilities calculated for holes 15+60P and 19+10P were both less than 0.1 feet per day so grouting of both holes was deferred. As shown by the non-uniform grout takes encountered throughout the foundation grouting program, these two grout holes probably do not give a true representation of Zone III subsurface conditions. The presence of isolated high grout take areas in Zones I and II, and higher rock mass permeabilities calculated for certain pressure test intervals in the deeper portions of preconstruction core holes DD-14 through DD-16 indicate that similiar foundation conditions would probably have been encountered had the entire lowest zone been grouted. The expected increase in rock competency with depth would have probably contributed to an overall higher rock mass permeability and average grout take for the Zone III rock.

4.99 The major high take areas encountered during east side grouting operations were all situated in granite bedrock, according to the foundation geology maps and/or exploratory core holes. The unusual high takes evident in Zone I between stations 16+05 and 16+35 probably indicate a localized network of permeable fractures present in more competent subsurface rock. An inspection of preconstruction test trench TT82-1, downstream of the grout curtain, revealed numerous calcium carbonate lined joints as well as several prominent open joints in generally moderately hard and moderately weathered bedrock (see pls. 13 and 31). Exploratory core holes 102C and 103C, drilled to investigate subsurface conditions in localized areas of the Zone II grout curtain between stations 12+80 and 13+55, encountered moderately to highly fractured and slightly to moderately weathered granite. A comparison with core holes 100C and 101C drilled in andesite bedrock revealed more direct evidence of grout filled fractures plus a higher composite RQD (36 percent) for the east side exploratory holes. However, circulation losses, ranging from 70 to 100 percent, were more evident during drilling, particularly in hole 103C. It appears that even with the implementation of an extensive single-line stage grouting program, voids and permeable fractures still existed in the foundation which required the additional injection of approximately 130 sacks during core hole grouting, and almost 25 sacks during supplemental foundation grouting.

4.100 Exploratory core hole 104C, drilled to investigate the major shear zone upstream of the grout curtain, revealed an approximate 30 foot cap of soft to moderately hard, moderately weathered to decomposed, sheared diorite overlying harder, less weathered sheared bedrock. Pressure testing indicated the 46-foot hole was very tight, with a calculated rock mass permeability of 0 feet per day. These subsurface conditions, if prevalent throughout the shear zone, would also create an effective natural barrier to underseepage extending down to at least the bottom depth of the Zone II grout holes.

4.101 GROUTING QUANTITIES. Records of quantities for foundation drilling and grouting were kept and agreed to on a daily basis throughout the job so the final totals could be easily and accurately tabulated. The estimated and actual amounts were as follows:

		Estimated Quantity	Actual Quantity
a.	Mobilization/demobilization	1 Job	1 Job
b.	Drilling exploratory grout holes	350 LF	266 LF
c.	Drilling grout holes	9,100 LF	10,535 LF
	Redrilling grout holes	-	25 LF
d.	Pipe for grout holes	500 LF	0 LF
e.	Drill set-ups		
	(1) Grout holes	370 EA	376 EA
	(2) Exploratory grout holes	4 EA	5 EA
f.	Pressure testing	175 HR	47.5 HR
g.	Grout pump connections	375 EA	213 EA
h.	Placing grout	2,500 sacks	2,058.1 sacks
	Waste grout	-	162.8 sacks

4.102 A comparison between the estimated versus actual quantities indicated that overall foundation conditions were substantially as anticipated. In some cases, a direct comparison could not be made without first normalizing the design figures to reflect the elimination of Zone III grouting on the east side. The analysis revealed the following: (1) the average grout take for the entire job (see table 7) was approximately 0.2 sacks per linear foot of hole, less than the 0.3 sacks per foot originally estimated; (2) primary grout hole footage accounted for about 80 percent of the required footage drilled, virtually the same as that estimated; (3) split-spaced grout holes and related grout hole set-ups accounted for approximately 35 percent of the actual totals, instead of the estimated 20 percent; and (4) the overage in actual drilling footage was primarily due to the unanticipated extension of the foundation grouting limits.

4.103 A review of the daily grouting quantity records for the job was made to determine the effects of percussion drilling equipment on design estimates regarding rates of drilling and grouting per shift. In general, it was originally anticipated that two 8-hour shifts per day for a period of approximately 4 months would be necessary to satisfactorily complete the grouting program in a timely manner. This estimate was based on the assumption that the subcontractor would use up to 3 slower production rotary drill rigs and 1 to 2 grout plants. The daily records indicated that the use of percussion drilling techniques, in conjunction with overall lower grout takes, contributed to the reduction in time necessary to complete the grouting program. The work was efficiently accomplished in 57 single-shift days. Employing usually one, but sometimes two, air-track drills, Jaques required only 39 work days to complete the 10,535 linear feet of grout hole drilling. This resulted in an average production rate of 270 feet (or approximately 11 single zone grout holes) per day. A total of 46 work days and 1 to 2 grout

plants were necessary in which to place the 2,058 sacks of grout. This resulted in an average daily grout placement rate of 45 sacks. Using the average calculated grout take of 0.2 sacks per foot, this meant 9 holes or zones could be grouted to completion in one shift.

4.104 SUMMARY. An analysis of the grouting data presented in table 7 indicates that although subsurface geologic conditions were different on each side of the valley, the results of the east and west side grouting programs were quite similar. The Zone I near surface rock on both sides proved to be relatively impermeable despite its highly fractured nature. The pervasive clay infilling in the Stage I core trench helped contribute to an average Zone I grout take in the andesite rock of 0.08 sacks per linear foot of hole. The near surface zone of weathering common in the basement complex rock was the primary reason the upper zone averaged only 0.07 sacks of grout per linear foot of hole. The average grout take for Zone II rock on both sides, approximately 0.2 sacks per foot, was much higher than for Zone I due to the presence of cleaner, more open fractures with more significant lateral extent. The same rationale could be used for the higher grout take calculated for the Zone III andesite rock. Undoubtedly the Zone III granitic rock would have also reflected an overall increase in permeability had the grouting of this zone been carried to completion.

4.105 CONCLUSIONS. The grouting program at New River Dam was deemed successful in view of the fact that its two stated objectives (see para. 4.70) were accomplished. However, judging from the low grout takes common in the upper zone on both the east and west sides, grouting probably did not significantly improve the foundation conditions which naturally existed, except in localized high take areas. In the lower zones, the stage grouting methods employed undoubtedly created a more impermeable barrier to unseepage. However, it is possible, based on the non uniformity of grout takes, that localized ungrouted permeable fractures may still exist at depth below the foundation surface. Constructing a staggered, multiple row grout curtain with an even tighter primary hole spacing would have reduced the likelihood of this occurring but the costs associated with such a program would have been out of proportion with the primary function of the dam. In addition, such alternatives were not considered necessary for a structure without a positive cut-off.

Outlet Works

DESCRIPTION

4.106 The outlet works for New River Dam, located approximately 130 feet west of the toe of the left abutment, consists of the following features: (1) an approach channel, trapezoidal in cross-section with side slopes of 1V on 2H and a base width of 40 feet; (2) an open rectangular concrete intake structure; (3) a 433-foot long cut and cover rectangular concrete conduit, 6.25 feet wide by 9.5 feet high; (4) an open rectangular concrete energy dissipator, which flairs out from 6.25 feet to 31 feet and drops approximately 17 feet in invert elevation, and which is surrounded by a grouted stone trapezoidal stilling basin; and (5) an outlet channel, trapezoidal in cross section with side

slopes of 1V on 2.5H and a base width decreasing from 31 feet to 14 feet, which terminates in a trapezoidal grouted stone section with a grouted stone cutoff. A portion of the approach channel is lined with grouted stone and the outlet channel lined with both grouted stone and ungrouted stone for erosion protection. The outlet conduit is capable of releasing up to 2665 ft^3/s when the water surface is at spillway crest. The energy dissipator at the downstream end of the conduit is designed to reduce the velocity of discharge prior to entering the natural stream. The outlet works plan, profile, cross sections and details are shown on plates 60 through 64.

GEOLOGY

4.107 The outlet works intake structure, conduit, and energy dissipator are founded on bedrock of the Precambrian basement complex. The bottom of the excavation for each of these concrete structures was mapped at a scale of 1 inch equals 10 feet. The foundation geology is presented on plates 37 through 39. The approach channel excavation was not mapped but is founded along most of its length in granitic bedrock. The outlet channel excavation was also not mapped but it founded in granitic bedrock between stations 14+91.5 and approximately 13+60, and poorly- to well-cemented alluvial streambed deposits (Qoal) downstream of station 13+60. Since the rock units exposed in the outlet works excavation are part of the basement complex, a separate discussion of individual rock units and geologic structure is not necessary. Instead, the general foundation conditions for each concrete structure will be described.

Intake Structure

4.108 The characteristics of the foundation bedrock in the intake structure (pl. 37) excavation vary considerably (photo 75). The left side of the excavation is mostly hard, competent granite while the right side of the excavation is a combination of moderately soft to hard, slightly to moderately weathered quartz diorite and soft to moderately soft, highly weathered diorite. A prominent 2 to 2-1/2-foot wide shear zone, trending N55°W and dipping 25°SW and filled with highly weathered to decomposed granitic bedrock and slickensided red clay gouge, extends from the toe of the slope on the left side of the excavation at station 20+77 diagonally across the bottom to at least station 21+25 on the right side of the approach channel. Approximately 1 to 2 feet of soft gouge and bedrock was excavated from the shear zone and the depression backfilled with 1-1/2 cubic yards of excess structural concrete from the conduit.

Conduit

4.109 The foundation bedrock in the outlet conduit trench (pls. 37 and 38) between stations 20+69 and 17+55 is composed of irregularly shaped masses of granite, quartz diorite, and diorite which reflect the complex diversity of the basement rocks. The bedrock is typically hard, slightly to moderately weathered and moderately to highly fractured. Dikes of weathered, generally shattered to brecciated intrusive igneous rock and thin shears and shear zones which trend generally in a northeast direction, obliquely to perpendicular to the conduit axis, and dip steeply downstream, frecuently dissect the basement

rock in this reach. The major shear zone exposed in the upstream transition zone of the dam excavation, except in the vicinity of the outlet works, apparently crosses the conduit excavation at station 19+05. The foundation bedrock between stations 17+55 and 16+35, except for one large pod of intrusive igneous rock, is hard, slightly weathered, moderately to highly fractured, and mostly tightly jointed granite. Large angular rock blocks are quite prominent between stations 17+55 and 17+25 (photo 76).

<u>**Energy Dissipator**</u>

4.110 The energy dissipator foundation (pls. 38 and 39) is composed of basement complex rock of variable hardness and degrees of weathering. Between the outlet conduit and approximately station 16+00, the foundation continues to be in the hard, competent granite. However, downstream of station 16+00, softer and more weathered bedrock is present, despite the greater depth of excavation. The parabolic slope is composed mostly of irregularly shaped layers of quartz diorite and diorite with inclusions and dikes of the intrusive igneous rock while the adjacent flat bottom section is composed mostly of hard, loosely keyed granite blocks in a decomposed granite matrix. This variation in rock quality resulted in significant overexcavation of portions of the energy dissipator (photo 77), necessitating the placement of 183 cubic yards of dental concrete, at the Contractor's own expense, to re-establish the "B" line elevation.

EXCAVATION

4.111 The Contractor used both mechanical and blasting methods to excavate approximately 42,000 cubic yards of material from the outlet works. Excavation of alluvium was generally done with scrapers while bedrock excavation was accomplished by either ripping with dozers, scraping with the 235 excavator, or blasting. Excavation for the intake structure and outlet conduit was performed using a combination of ripping and scraping while the bedrock in the deeper energy dissipator and outlet channel excavations was loosened by blasting. Blasting was performed without the benefit of a demonstration of the blasting technique to be utilized as required by the contract specifications so the Contractor was solely responsible for producing an acceptable end product. Final slope trimming in the approach channel and outlet channel excavations was accomplished by either a dozer with a slopeboard or a motorgrader. None of the excavated bedrock was considered suitable for use on the project so it was hauled by scrapers and rock trucks to the miscellaneous fill area for disposal. No significant foundation problems were encountered which required additional Government directed excavation to be performed. The soft, weathered bedrock encountered at invert grade within portions of the energy dissipator and intake structure was considered adequate to serve as a foundation for the concrete structures.

<u>**Mechanical Excavation**</u>

4.112 Excavation for the outlet works concrete structures commenced on 12 January 1984 and was completed approximately 1 month later in February 1984. A D9H dozer equipped with a double shank began ripping the outlet conduit trench near station 20+40 once the Stage II core trench excavation had

progressed past dam station 14+40, the approximate edge of the conduit excavation. The work proceeded rapidly and smoothly although the Contractor had difficulty ripping the hard granite in the downstream portion of the conduit excavation. The Contractor's methodology for excavation was similar to that employed for the left abutment. Ideally, the dozer would rip the bedrock down to approximately 1 foot above "A" line and then to protect the integrity of the foundation surface, the 235 excavator equipped with a scraper bar would remove material down to invert grade (photo 78). This procedure would have been completely successful if it were not for a grade check error discovered after foundation preparation which required additional excavation using the dozer. Localized high spots within the conduit trench, as well as the entire width between stations 20+40 and 19+60 had to be removed down to grade (photo 79). A subsequent grade check after foundation preparation revealed only a few small high spots which were removed by jackhammer. However, it now appeared the invert elevation of the conduit trench was now up to approximately 1 foot below "B" line in numerous areas. Operation of the heavy tracked dozer on the highly fractured foundation surface was probably the primary contributor to overexcavation although removal of excess bedrock during subsequent high pressure air cleaning may have also been a factor.

4.113 The excavation for the remaining upstream portion of the outlet conduit and the intake structure was accomplished using the 235 excavator. Once the rough excavation was complete, the more detailed work, which included the wing-wall steps and upstream cut-off trench, was done using a small Case backhoe, jackhammers, and a D8K dozer with a single tooth ripper (photo 80). The contrasting foundation conditions encountered on both sides of the intake structure required different methods to finish the excavation down to grade. On the left (east) side, the Contractor had little success ripping the hard competent granite in the cut-off wall excavation so jackhammers were used to remove high spots above grade. In addition, jackhammers were also used on the wing wall foundation when continued excavation with the small backhoe proved ineffective. On the right (west) side, the softer, more weathered diorite was extremely easy to excavate with the Case backhoe. In fact, the bottom step of the 3-step tier designed to reduce the amount of bedrock excavation for the wing wall foundation was eliminated to provide a more adequate foundation for the concrete structure. The variable weathering and hardness characteristics of the bedrock precluded excavation of the intake structure cut-off trench and adjacent approach channel area to the specified lines and grades. The typical configuration of the trench excavation is shown on plate 64. The resultant overexcavated area between the invert concrete slab and the approach channel (photo 81), together with the upstream wingwall backfill areas, were subsequently backfilled with 90 cubic yards of lean (dental) concrete (photo 82). The concrete was wrapped around the ends of both wingwalls to provide additional erosion protection.

4.114 Mechanical methods were also used to excavate the approach channel and downstream portion of the outlet channel. The granitic bedrock present in most of the approach channel excavation, except in the immediate vicinity of borrow area no. 3, was generally ripped down to grade. However, the entire width of the channel between stations 22+50 and 24+10 required light blasting for effective excavation. Shallow blast holes were drilled to an average

depth of 5 feet on an approximately 6 x 7-foot pattern. Since bedrock was not encountered in the outlet channel excavation downstream of station 13+60, that portion of the channel was excavated using scrapers.

Production Blasting

4.115 Since the D9H dozer was having difficulty ripping the hard granite bedrock in the outlet conduit trench, the Contractor decided it would be more cost-effective to blast the similarly hard granite exposed by the excavator in the energy dissipator area (photo 83). The Contractor elected to proceed directly with production blasting without first demonstrating the blasting technique to be utilized as required by the contract specifications. The Government approved their proposal with the understanding that any overexcavation resulting from blasting would be solely the Contractor's responsibility and would be replaced with concrete at no additional cost to the Government. A "step-drilling" plan, similar to the one used effectively in the spillway, was employed to blast the bedrock. Three production blasts (nos. 5, 6 and 7) were required between station 16+36 in the energy dissipator and station 13+64 in the outlet channel. According to the blasting summary in table 6, a total of 5185 cubic yards of rock (approximately 12 percent of the total outlet works excavation) were blasted using 3387 pounds of ANFO explosive. This resulted in an average powder factor of 0.65 pounds of explosive per cubic yard of rock.

4.116 For shot no. 5, between stations 16+36 and 15+16, blast holes were drilled about 6 inches below "A" line on 3 hole patterns (6 x 6, 6 x 7, and 5 x 6 feet) due to the varying shape of the dissipator. Controlled blasting techniques were not utilized within the buffer zones shown on plate 62. Hole depths varied between 1-1/2 and 20 feet, depending on invert grade, and "soft" rock zones encountered in the bottom of numerous blast holes were not logged or deck loaded accordingly. These soft zones at invert grade, coupled with the lack of adequate grade control, caused significant problems during excavation, as the large tracked backhoe overexcavated portions of the shot, particularly on the slope, by as much as 4 feet (photo 77). Although it was doubtful that blast effects damaged the rock below "B" line elevation, the Contractor nonetheless drilled blast holes only to "A" line for production shot no. 6, between stations 15+04 and 14+20; and for production shot no. 7, between stations 14+13 and 13+64. In addition, each hole for shot no. 6 was logged and all soft zones at invert grade delineated. During excavation, a grade checker was present to insure that excavation with the excavator was carried no deeper than "A" line. Blast holes for shot no. 6 were drilled to depths ranging from 4 to 18 feet on the same 3 hole patterns utilized for shot no. 5. Prior to excavation of shot no. 6, the remaining short section of bedrock in the outlet channel excavation was blasted to allow both areas to be excavated simultaneously. For shot no. 7, only 26 holes, on 7-foot centers, were drilled to average depths of 7 feet.

FOUNDATION PREPARATION AND TREATMENT

Surface Preparation

4.117 Only those portions of the bedrock foundation on which concrete structures were to be placed required formal foundation preparation and treatment. Within the conduit and intake structure excavations, a Case backhoe equipped with a scraper bar was used to initially scrape off loose material from the foundation surface (photo 84). This "coarse" cleaning was followed by a more detailed "fine" cleaning using low pressure air blasting, shovels and hand picking. Cleaning of the low angle shear zone crossing the upstream invert section of the intake structure was accomplished by removing approximately 1 to 2 feet of clay gouge and decomposed bedrock down to the underlying hard, more competent granitic bedrock. This left a near vertical bedrock face extending below grade on the downstream side of the shear zone, against which dental concrete was placed to provide a more level foundation surface.

4.118 In the energy dissipator excavation, it soon became apparent that normal foundation preparation methods were causing continued degradation of the bedrock surface, particularly in the bottom section. Both high and low pressure air blasting were blowing away the soft, highly weathered granite matrix surrounding the hard granite blocks, causing the blocks to work loose (photo 85). The following procedures were subsequently established for cleaning the dissipator bottom: (1) high pressure air blasting only and not mechanical equipment would be used to quickly expose the bedrock surface in areas not previously cleaned; (2) once bedrock was exposed, very low pressure air would be used to lightly and quickly blow out all loose rubble to avoid undue degradation; (3) all remaining loose material would be removed by hand picking or using whisk brooms if necessary; and (4) cleaning should proceed in a downstream direction to avoid repeated foot traffic over previously cleaned areas.

Surface Treatment

4.119 Once foundation preparation was completed in any particular section, the foundation treatment required was in the form of dental concrete placed as backfill in overexcavated areas. No grout slurry was required because the joint structures were generally tight or well healed. Immediately prior to concrete placement, any small pockets of loose rock were hand picked from the bedrock surface and the surface wetted. In the outlet conduit trench, it was necessary to replace the overexcavated areas with concrete as required in the contract specifications. The Government assumed responsibility for payment because of the variable physical characteristics of the foundation bedrock. The Contractor placed 82 cubic yards of dental concrete within the conduit trench between stations 20+40 and 16+35. The concrete was placed only within the limits of the conduit section; no concrete was required in the areas to be backfilled with compacted transition material. Most of the concrete, particularly in the reach between stations 20+40 and 19+90, and downstream of station 17+95, was placed as a leveling slab to bring the conduit invert up to "B" line elevation (photo 86). Localized depressions were also backfilled to create a level invert surface. Outside the limits of the core trench

excavation, the concete was placed directly from the truck chute. Within the core trench excavation, a crane and hoist bucket were used due to the lack of equipment access to the conduit excavation. After placement, the concrete was vibrated and then screed to a uniform "B" line elevation in the leveling slab areas. In the energy dissipator, the Contractor assumed responsibility for placing dental concrete in overexcavated areas, including the entire invert slope, to bring the foundation back up to design grade (photo 87). Approximately 183 cubic yards of concrete were placed, using a concrete bucket and crane.

Final Cleanup

4.120 Once any required foundation treatment was completed, wooden bulkheads for the various concrete structures were constructed. Following installation of the invert section reinforcing steel, the bedrock or concrete surfaces were given a final cleaning prior to wetting and placement of structural concrete. Most of the cleaning could be accomplished simply by low pressure air blasting through the steel mats directed toward the backfill areas (photo 88). However, in the bottom section of the energy dissipator, where construction activities had resulted in additional significant degradation of the bedrock surface, air blasting alone was not sufficient to remove loose rock and other debris from the irregular depression-filled bedrock surface. Laborers had to crawl underneath and between the steel mats and hand pick loose rubble and rock blocks from the floor of the trench (photo 89). The work was difficult but was accomplished in a satisfactory manner. It was virtually impossible, given the foundation conditions and difficult access, to remove all loose material from certain bedrock depressions. However, since most of the areas in question were below "B" line elevation, it was felt the small amount of unsuitable material in localized areas would not adversely affect the integrity of the invert slab.

CONCRETE PLUG

4.121 Between January and March 1984, the outlet works conduit was under construction. However, due to the extended water curing program for the conduit concrete, which was initiated to enable the concrete to attain the 4000 lbs/in^2 compressive strength criteria established in the contract specifications, placement of the lean-mix plug was delayed until 25 April 1984. The purpose of the plug was to preclude differential settlement and cracking of the core zone over the conduit, as well as to inhibit seepage along the exterior of the box. The concrete plug was not constructed as designed due to the greater depth of excavation in the adjacent core trench (photo 90). Between outlet works stations 18+03 and 18+80, the limits of the plug; the elevation difference between the core trench excavation and the outlet excavation ranged from approximately 0.5 feet to 6.5 feet, with the average difference being about 2.5 feet. This was much less than the typical 6 feet shown on plate 62 of the contract drawings. Instead of being confined within the limits of the conduit trench, the plug, as constructed, extended from the top of the conduit on a 1V on 1H slope out over the core trench surface (pl. 62); to approximately dam station 14+12 on the east side and dam station 14+46 on the west side. A total of about 485 cubic yards of concrete were placed.

4.122 In preparation for concrete placement, the conduit surface within the area of the plug was sandblasted to remove the curing compound and to make the concrete suitable for bonding with the lean-mix plug. The bedrock surfaces within the limits of the plug were then air cleaned and handpicked of all loose rock and foreign matter. Since the configuration of the concrete plug was different than originally anticipated, contact grouting along the outlet conduit trench walls was not considered appropriate because the elevation differences between the floor of the core trench and the floor of the conduit trench were so minimal. Instead a 2-foot wide by 4-inch high grout slurry cap was placed on prewetted bedrock along the embankment centerline between the conduit and side walls of the trench. Approximately 5.3 cubic feet of grout consisting of 1 part sand to one part cement was mixed by hand and placed using the crane bucket. It was hoped the slurry cap would act as a waterstop and inhibit seepage along the rock-slurry contact. To insure a good bond between the grout cap and concrete plug, the remaining bedrock was wetted and concrete placement commenced before the grout slurry had a chance to completely dry.

4.123 The Contractor elected to place the concrete plug without the benefit of bulkheads for containment. To minimize placement problems, the water content of the concrete was frequently varied to allow placement on a 1V on 1H slope. A high slump concrete was generally placed at the rock contact to allow better penetration into the rock, while a lower slump concrete was generally placed on the slope to allow the uncontained mass to remain stable. On the east side of the conduit, concrete placement was accomplished strictly with the concrete bucket and crane (photo 91), while on the west side, the concrete was placed either using the bucket or directly from the truck chute. Placement on the west side was much easier to control due to slower rate of placement from the chute and the more level bedrock surface. On the east side, there was a tendency initially for the concrete dumped from the bucket to flow past the limits of the plug onto bedrock surfaces not acceptably cleaned. To consolidate and insure bonding, the concrete was vibrated in place with special emphasis along the conduit and the contact with the foundation. The upper portion of the plug was constructed slightly steeper than 1V on 1H to eliminate the feather edges at the contact with the top of the conduit (photo 92). After concrete placement was completed (photo 93), the Contractor was instructed to remove all excess and loose concrete along the base of the plug and to trim back all "feather" edges.

CONDUIT BACKFILL

4.124 The remainder of the outlet conduit trench was backfilled with transition materials. The floor and side walls of the trench were cleaned using shovels, a Case backhoe and low pressure air blasting before any materials were placed. Transition materials were placed in 4-inch loose lifts and compacted with a small double-drum vibratory roller or hand held power tampers. The materials were placed using a front-end loader and spread using shovels to the desired lift thickness to avoid having any plus 3-inch rock come in contact with the conduit. Before each lift was placed, the preceding lift was scarified by a small lawn tractor equipped with a rake.

4.125 The same foundation cleaning and material placement procedures were used for the energy dissipator and intake structure backfill areas. However, the excavation between the wing walls and the approach channel sideslopes was backfilled with lean concrete for erosion protection.

APPROACH CHANNEL GROUTED STONE

4.126 After the outlet works was completed, subsequent inspections by Engineering Division personnel indicated the need for grouted stonework within the approach channel to prevent future erosion and to provide additional structural integrity of the intake structure. The work was accomplished under Modification of Contract P00009 at a total negotiated cost of $24,779. The change included the placement of an 18-inch thick section of grouted stone from about station 21+00 to station 21+30, with a 36-inch grouted stone cutoff at station 21+30. To effectively stabilize the approach channel, both the channel floor and sideslopes, including the 10-foot wide berm above the lower sideslope on the left (east) side, were covered (see pl. 60 and photo 9).

Spillway

DESCRIPTION

4.127 The spillway for New River Dam is excavated in bedrock through a natural topographic saddle approximately 700 feet northwest of the right abutment of the dam (photo 10). It was designed to retain the SPF while providing a controlled discharge of the PMF. The spillway, in conjunction with the outlet works, will pass a peak discharge of 33,000 ft^3/s with 5.6 feet of freeboard. The spillway is trapezoidal in cross-section, with lower side slopes of 1V on 0.5H and upper side slopes of 1V on 1H, separated by horizontal benches 12 feet wide at a height of 30 feet above invert. A crest elevation of 1456.2 feet was established for the concrete sill at station 17+50. A 80-foot maximum cut and excavation of approximately 100,000 cubic yards of material was required to reach invert elevation. The spillway is approximately 1325 feet in length with an average base width of 75 feet. Irregularities in the natural rock slopes resulted in varied bottom and bench widths. Determination of original side slope angles was based upon experience in similar rock. However, during excavation, the upper 1V on 0.75H side slopes were flattened to 1V on 1H due to safety considerations. The benches were designed to reduce the velocity of surface runoff down the slopes and to catch slope talus before it reached the bottom. The spillway is unlined except for the control section at station 17+50, which is stabilized by a concrete sill across the bottom and up the side slopes to the bench. The spillway plan, profile, and cross sections are shown on plates 58 and 59.

GEOLOGY

4.128 The geologic conditions encountered at the spillway were essentially the same as those anticipated from the pre-construction investigations. However, close examination of the spillway excavation and detailed petrographic analyses of selected rock samples resulted in an expanded and somewhat revised rock classification scheme and a reinterpretation of the geologic structure of the spillway area. The various lithologic units are described separately followed by a discussion of the geologic structure.

Lithologic Units

4.129 Both walls of the spillway excavation were mapped at a scale of 1 inch equals 15 feet. However, only the geology of the north (right) wall (plates 40 through 42), which is representative of the south (left) wall, is included in this report. The floor of the excavation was never thoroughly cleaned so it was not mapped. For mapping purposes, the layered volcanic sequence exposed in the spillway excavation was divided into 3 major units: (1) tuff, (2) flow breccia, and (3) andesite. The tuff unit was further subdivided into 4 members based on macroscopic structural and textural characteristics supplemented by detailed petrographic work. The tuff unit is composed of an upper ash-fall member, intermediate ash-flow and ash-fall members, and a lower or basal ash-fall member. The names for the different lithologic units were taken from rock classifications derived from petrographic analyses. The rock unit originally identified as tuffaceous sandstone on plates 2, 19, 20, and 22 of the contract drawings was subsequently re-classified as an ash-fall tuff. However, the sandstone exposed in the test trenches (see pl. 19) downstream of the spillway excavation can probably still be classified as a deposit of some ancient sedimentary environment and not a product of a volcanic episode. All of the volcanic rocks in the spillway were also assigned a Tertiary age based on information from Eberly and Stanley (1978).

4.130 TUFF. Apparently the youngest, although topographically the lowest, rock unit in the spillway is the approximately 70-foot thick tuff sequence, near the downstream end of the spillway excavation between stations 12+30 and 13+05. The tuff unit was probably formed by the accumulation of layers of pyroclastic material derived from a series of explosive volcanic eruptions. Four tuff members were identified in the excavation and are described separately as follows.

a. <u>Tvt_1</u>. The upper, least exposed and farthest downstream member of the tuff sequence was classified as an ash-fall tuff (Tvt_1). It is a crudely to well stratified assemblage of well sorted fine to coarse grained ash consisting of dark purple to red glass and andesite fragments, pink devitrified pumice fragments and minor phenocrysts of feldspar and quartz. The exposed rock tends to be moderately soft to moderately hard and moderately to highly weathered. An air-fall origin for this rock can be implied due to the well-sorted nature of the deposit. In addition, the texture indicates this member has not undergone compaction and welding as have the other members.

b. <u>Tvt_2</u>. Following in sequence is a very small exposure of mottled salmon-pink to gray, moderately hard and slightly to moderately weathered lapilli ash-flow tuff (Tvt_2). This member consists of coarse ash (1/4 mm to 4 mm) and lapilli-size (4 mm to 21 mm) glass fragments, altered devitrified pumice fragments and rounded andesitic rock fragments in a finer grained ash matrix of similar composition but also including quartz, feldspar, and minor mafic minerals. Only slight welding of the component particles is evident. A flow origin for this rock is suggested by the following evidence: (1) the unsorted nature of the deposit, and (2) the heterogeneous nature of the lapilli fragments and ashy matrix.

c. Tvt$_3$. Following the ash flow tuff is another layer of ash-fall tuff (Tvt$_3$). This member, light purple in color, moderately hard to hard and generally unweathered, is a crudely stratified assemblage of fine to medium grained devitrified pumice fragments, subangular to rounded volcanic glass fragments, and quartz and feldspar phenocrysts, with minor amounts of volcanic rock fragments and mafic minerals. Welding of the component particles is slight to moderate, as indicated by the presence of dark gray to black elongated layers of pumice fragments interbedded with layers of pinkish to purple less consolidated pumice fragments. The crude stratification is the result of the degree of welding and the grain size of the pumice fragments.

d. Tvt$_4$. The lower, but stratigraphically highest, member of the tuff sequence represents a moderately welded ash-fall tuff (Tvt$_4$), consisting of welded pumice fragments with subordinate obsidian glass fragments and phenocrysts of feldspar, mafic minerals, and minor quartz. The tuff is light purple to purplish-red in color, hard, generally unweathered and well stratified due to the moderate degree of welding. Welding has also resulted in the rock taking on the appearance of a compact glass or vitrophyre.

4.131 FLOW BRECCIA. The approximately 20-foot thick flow breccia (Tvfb) unit, exposed in the spillway excavation between stations 13+05 and 13+35, may represent a strongly fragmented flow which underwent extreme brecciation at the time of emplacement. The rock is reddish-brown in color and consists of angular to subrounded fragments and blocks of highly vesicular andesite and basalt rock up to 2-foot diameter in a reddish cindery matrix. There is no preferred orientation of the rock fragments or blocks or welding of the matrix material. Secondary mineralization within the rock has occurred, with zeolite deposits filling numerous vesicles. The upper 1 to 7 feet of the present surface of the flow is soft, friable and highly weathered while the remainder of the exposed flow tends to be hard, massive and moderately weathered. A thin layer of flow breccia, the near surface of which is mixed with soil, slope wash and caliche, also caps the andesite bedrock on the upstream side of the spillway between stations 18+90 and 19+50 and locally infills open joints in the bedrock. This flow is probably contemporaneous with the downstream breccia unit. The composition of the flows appear to be similar although the color of the upstream flow tends to be more subdued due to weathering effects.

4.132 ANDESITE. Upstream of station 13+35, the bulk of the spillway excavation was in andesite (Tva) rock similar to that exposed in the right abutment and Stage I core trench excavations. Since the composition of the two flows are the same, the detailed rock description for the dam foundation andesite is applicable to the spillway andesite. Although topographically the highest rock unit, the andesite is apparently the oldest rock unit present in the spillway excavation. The rock is characteristically medium to dark gray in color, hard, blocky to platy, highly jointed, and unweathered. The upper 1 to 4 feet of the andesite near the flow breccia contacts is brownish-purple to mottled purple-maroon-gray in color, more vesicular, slightly to moderately weathered, and locally brecciated and rehealed. These distinct color changes may be related to different concentrations of minerals in the rock mass, secondary weathering, or possible reheating by the flow breccia unit. The andesite required both mechanical and blasting techniques for effective excavation. The typically highly fractured, loosely keyed upper part of the

block flow was rippable with a D9H dozer while the remainder was blasted because the more irregular joint spacings produced locally more massive, resistant outcrops which could not be effectively removed by mechanical means. The soft, friable flow breccia was easily ripped while the moderately to highly fractured tuff and massive flow breccia required blasting for effective excavation.

4.133 ALLUVIUM. Downstream of station 12+50, the spillway excavation is predominantly in alluvium (Qoal), consisting of residual soil and andesite slopewash with an underlying 1 to 3 foot thick zone of well indurated caliche cemented rock fragments on top of tuff bedrock. The alluvial cover in the remainder of the spillway is generally thin (usually less than 1 foot thick) and sporadic and includes colluvial deposits, consisting of loose, desert varnished andesite blocks and rubble locally capping the mountains in the saddle area.

Geologic Structure

4.134 The spillway excavation consists of a layered volcanic sequence of probable Tertiary-age which strikes approximately N50°W and dips approximately 30°NE (photo 94). Petrographic evidence appears to indicate the contacts between the various rock units have become oversteepened, possibly the result of uplift and tilting during the period of Basin and Range tectonism, and that the exposed rock actually represents an inverted sequence. This assumption is based on the fact that the tuff sequence shows a general trend from a poorly compacted and welded deposit (Tvt_1) to a well compacted and stratified, moderately welded, glassy deposit (Tvt_4). Since welding is often caused by the high temperature of the tuff on deposition, in conjunction with the weight of the overlying material, it appears the welded ash-fall tuff which lies above the other tuff members was actually the first in a series of pyroclastic deposits.

4.135 The flow breccia, as noted before, may represent a strongly fragmented flow which underwent extreme brecciation at the time of emplacement. It may also represent the leading edge or the upper surface of a highly viscous andesite flow. The geologic log of drill hole DD-37 on plate 22 appears to indicate a major gradational change from unaltered gray andesite through mottled, altered partially brecciated and rehealed andesite to massive, scoriaceous flow breccia.

4.136 The viscous nature of the andesite flow probably resulted in the lava developing a strongly jointed and platy to blocky structure (photo 95). The joint structures are typically well developed, open, closely to moderately spaced (1 to 12 inches), and arranged in subparallel to wedge-shaped patterns. A secondary sinusoidal pattern of steeply-dipping joints is also present, most notably in the core of the excavation between stations 17+00 and 18+00 (plate 40). Like the andesite rock underlying the dam embankment, the joint orientations are quite varied, although strong patterns are recognizable in localized areas of the excavation (see pls. 41 and 42, and photo 96). A stereonet plot of joint attitudes measured in the spillway excavation revealed only 1 broad joint system with an average strike of N85°E and an average dip of 80°NW. However, within this system, joint orientations ranged from N80°W, 55°SW to N45°E, 40°NW.

4.137 The joint surfaces are generally planar to arcuate, smooth, and normally clean. Infilling material, consisting of calcium carbonate, and occasional rehealed breccia, along with minor black oxide staining, is frequently present. However, joint planes in a near surface "zone of weathering", which extends down to a maximum depth of about 15 feet in the saddle area, are predominantly soil stained or soil infilled. The characteristically open joints, as well as joint planes adversely oriented toward the excavation, have caused locally unstable conditions to develop in the rock mass, resulting in frequent rockfalls and bench widths narrower than the specified 12 feet in some areas. The flow layering which trends N40°W and dips 30°NE, and gives the andesite a noticeably platy character when distinct, generally parallels the rock contacts.

4.138 No faults were noted in the andesite block flow. However, one small normal fault, striking N5°W and dipping 65°SW, was found to have caused a 4-foot displacement between tuff members Tvt_3 and Tvt_4 at approximately station 12+70. This fault may have been the result of displacement related to volcanism or to later tectonism.

EXCAVATION

4.139 Since the Contractor planned on using some of the excavated spillway material during construction, he began excavation soon after mobilizing to the site. Excavation commenced on 24 October 1983, and continued sporadically until December 1984, with the necessity for material usage generally dictating the need for excavation. The Contractor removed approximately 100,000 cubic yards of material using a combination of ripping and blasting techniques. The Contractor's selection of the most suitable excavation technique at a particular time depended on: (1) the nature of the rock to be excavated, (2) the availability of equipment, (3) the end-product use of the excavated material, and (4) the logistics. None of the excavated material was considered suitable for use as stone protection or in critical areas of the dam or dike 1 embankment fills; the flow breccia and tuff, because of its variable physical characteristics and small quantity available, and the andesite, because of its potential to undergo extensive breakdown possibly due to thermal expansion and contraction. However, the Contractor was permitted to use the andesite as landscape stone, as slope protection for the diversion levees, and in the downstream pervious shell zone of the dam embankment.

4.140 The contract specifications relating to the spillway excavation were written to yield the best possible end-product given a certain set of field conditions and the Contractor's natural tendency to use the most cost-effective, although possibly unsuitable (from the Government's standpoint) excavation techniques. In this regard, buffer zones were specified adjacent to the final slope lines and above the spillway invert. Within these buffer zones, approved controlled blasting techniques or mechanical excavation techniques were required to produce relatively smooth and sound rock faces at the final excavation lines and to prevent damage to the rock outside the prescribed limits of excavation. Because of the Government's concern over the possible negative effects of the excavation methods utilized on the highly fractured rock mass, the Contractor conducted 5 demonstrations of proposed blasting techniques, and methods of grade control to obtain results satisfactory to the Government. All pertinent data from these demonstration shots as well as production shots are tabulated in table 6.

Mechanical Excavation

4.141 To effect equipment access to the spillway prior to the start of excavation, the Contractor pioneered an access road from the miscellaneous fill area to the saddle area via the upstream flank of the mountain. The Contractor elected to begin excavation in the saddle area by ripping, using a D9H dozer equipped with a double shank and slopeboard (photo 97). He was instructed to remain outside the buffer zones, and only after a satisfactory demonstration of slope grade control had been made with the slopeboard could excavation within the buffer zones proceed. After the excavation had progressed to a maximum depth of 10 feet (approximate elevation, 1521 feet), concerns were expressed by Contractor and Government construction personnel about the stability of the 1V on 0.75H design side slopes from a safety standpoint. They felt rockfalls would pose an increased safety hazard as the excavation progressed and strongly recommended flattening the side slopes to reduce the sloughing problem. It was eventually decided that the most economical and practical solution to the sloughing problem would be to change the upper side slopes from 1V on 0.75H to 1V on 1H. Any additional trimming would only lead to excessive encroachment into the adjacent hillsides. Future concerns over the stability of a 1V on 1H slope could be handled by the installation of wire mesh screening to control sloughing. Once an acceptable demonstration of slope grade control was made, the Contractor was permitted to extend his excavation into the buffer zones and subsequently to the final excavation lines. By early November 1983, the spillway had been completely excavated down to a maximum depth of 20 feet in the saddle area (approximate elevation, 1511 feet). Rock outcrops resistant to mechanical excavation were encountered on the south side of the spillway so the Contractor proposed to implement a blasting program to complete the remaining excavation above the bench.

Presplit Demonstrations

4.142 The Contractor's original blasting plan consisted of establishing 7-foot wide benches below the existing 1V on 1H side slopes and then presplitting on 1V on 0.75H slopes down to bench grade. He felt that presplitting on a 1V on 1H slope would be extremely difficult and would not produce suitable results. However, since Government construction personnel again expressed concerns over a steeper slope on the north side of the excavation, the Contractor was permitted to continue mechanical excavation on this side only after a determination was made that an asymmetrical spillway above the bench would not impact the design requirements.

4.143 The Contractor initially proposed to demonstrate only one presplit (controlled) blasting technique (blast no. 4) in an area containing "hard rock humps" of altered andesite and flow breccia which the dozer was having difficulty ripping. This location, between stations 18+70 and 18+82, was actually below bench grade. The plan called for drilling presplit holes on 3-foot centers along a 1V on 0.5H slope line perpendicular to the spillway centerline together with 2 rows of vertical reliever holes on a staggered 6 x 7-foot pattern. The presplit holes would be loaded with Atlas 40 percent Kleen Kut presplit powder and the burden holes with ANFO. The Government felt that this test area would not be representative of the bulk of the spillway

excavation and requested that the Contractor also demonstrate presplitting under more "normal" conditions: on a 1V on 0.75H slope in the highly fractured andesite above the bench and parallel to the spillway centerline. The Contractor agreed to a second presplit demonstration (blast no. 4a), which was situated in the saddle area between stations 17+25 and 17+43, on the south side of the excavation about 25 feet from the toe of the existing slope. In this test area, the same blast hole spacing and explosives would be used, except that the 3 rows of reliever holes were not drilled on a staggered pattern.

4.144 Both demonstration blasts occurred on 18 November 1983 and neither produced acceptable results. In both test areas it was apparent that the blast just shattered the rock around each presplit hole and did not initiate a presplit surface along the excavation line. Numerous large rock blocks remained virtually unaffected by the blast and still protruded into the excavation. The failure to produce a continuous crack between adjacent presplit holes probably allowed the ANFO-loaded reliever holes to loosen and otherwise damage the highly fractured rock at and beyond the presplit face. It was concluded that the presplitting technique as conducted was not suitable for use in the andesite bedrock. The presence of numerous open fractures probably dissipated the explosive energy from the blast. In addition, the presplit hole spacing may have been too wide for the small (2-1/2-inch) hole diameter. However, the biggest factors preventing successful presplitting probably were the variable joint orientations and the irregular joint spacings in the generally highly fractured rock mass.

4.145 After the two unsuccessful presplit demonstrations, no further test blasts were conducted until February 1984. However, by continued ripping and scraping, the Contractor was able to lower the bottom elevation of the spillway excavation in the saddle area to 1503 feet.

Step-Drilling Blasting Demonstration

4.146 In February 1984, the Contractor proposed completing the remaining excavation above the bench on a 1V on 1H slope utilizing a "step-drilling" blasting technique in lieu of presplitting (see fig. 9). The plan involved drilling offset vertical slope holes to a depth of about 1 foot above the final excavation line and vertical production holes down to the bench elevation on an 8 x 8-foot pattern. Each hole would be loaded with ANFO and a millisecond delay pattern used. The intent of using delays was to protect the slopes by producing a "free-face" in the middle of the excavation to which the explosive gases and muck could heave to. In addition, the maximum amount of explosive detonated at one time would be reduced. During excavation, final grade trimming would be accomplished using a dozer slopeboard. The third demonstration blast (blast no. 8) occurred on 9 February 1984 between stations 17+00 and 17+48, approximately 61 feet from the toe of the existing south wall. Although not a controlled blasting technique per se, the step-drilling plan did produce acceptable results so it was approved by the Government for use as an alternative to presplitting. The blast did not produce any overbreak in the highly fractured andesite rock mass but in fact generally left a 1 to 2-foot buffer in front of the excavation line which was then removed to grade with the slopeboard.

Upper Spillway Production Blasting

4.147 Only 1 production shot (blast no. 9), on 8 March 1984, was necessary to reach the approximate bench elevation of 1486 feet (photo 98). Using the approved blasting plan shown in figure 9, 27,217 cubic yards of rock were blasted between stations 15+00 and 18+00 using 21,550 pounds of ANFO and minimum 25 millisecond delays. This worked out to an average powder factor (PF) of 0.79 pounds of explosive per cubic yard of rock.

Step-Drill/Trim Blast Demonstrations

4.148 Once the excavation was completed down to the bench level, the Contractor had to once again demonstrate controlled blasting techniques on a 1V on 0.5H slope. Their proposal was similar to the unacceptable presplit techniques demonstrated earlier. Presplit holes were again drilled on 3-foot centers but the reliever and production holes were spaced on a wider 8 x 8-foot pattern. The presplit holes were loaded with Atlas 52 percent Kleen Kut powder while the remaining holes were loaded with ANFO. The results of the presplit demonstration (blast no. 10) on 19 June 1984 again proved unsatisfactory so the Contractor next proposed conducting two demonstrations in one. Two blasting techniques would be utilized; a step-drilling plan, similar to the approved blasting plan used above the bench, on the north side of the test area, and a trim blast plan on the south side of the test area. Both plans included production holes on an 8 x 8-foot pattern and slope holes on a 6 x 8-foot pattern. However, the trim blast plan also included a row of slope line holes drilled on 4-foot centers. Unlike presplitting, the trim line holes and slope holes would be shot last, after the main round had been fired. All vertical holes would be loaded with ANFO while the trim line holes would be loaded with Kleen Kut powder. The test area (blast no. 11), between stations 15+77 and 16+25, was shot on 27 June 1984. Due to the limited work area, both slopes were fully exposed using a Caterpillar 245 excavator and each technique evaluated. It was determined that, despite the rough slope trimming with the large backhoe, which resulted in occasional localized deviations from slope lines and grades, the trim blast produced satisfactory results under diverse geologic conditions (i.e., non-uniform joint orientations and spacings) and was therefore an acceptable controlled blasting technique to use. During production blasting, however, the Contractor would continue to use the slopeboard for tighter grade control.

Lower Spillway Production Blasting

4.149 The trim blast plan (see fig. 10), although approved for production blasting below the bench, was not acceptable for blasting of the four-foot horizontal buffer zone above the spillway invert or excavation of the spillway sill section. According to the contract specifications, blasting of the horizontal buffer zone was to be accomplished by either a tight 3 x 2 1/2-foot vertical hole pattern or other approved alternative spacing and blasting techniques, provided they were first demonstrated in an area above the buffer zone. However, a misunderstanding arose following approval of the trim blast plan. The Contractor assumed he could demonstrate removal of the actual buffer zone using a production shot incorporating the approved trim blast plan (blast no. 12) as an alternative technique. As a result, production holes on

an 8 x 8-foot pattern were drilled down to invert elevation (contrary to the specifications) and on 16 July 1984, the entire area between stations 18+00 and 19+10 was shot without Government approval. During a subsequent meeting between Contractor and Government representatives, the Contractor was allowed to continue production blasting but only above the horizontal buffer zone and could not remove the buffer zone before adequately demonstrating his ability to do so.

4.150 Using the typical configuration of the approved blast plan shown in figure 10, the Contractor blasted the remaining rock above the buffer zone between stations 13+25 and 18+00 using only two production shots (blast nos. 13 and 14 on 21 August and 27 August 1984, respectively). According to the blasting summary in table 6, 41,427 cubic yards of rock were blasted using 30,600 pounds of explosives and up to 12 delays. This resulted in an average PF of 0.74. Production hole patterns varied from 6 x 6-feet between stations 13+25 and 14+00 to 9 x 9-feet between stations 14+00 and 18+00. The production blasting left an approximate 8-foot horizontal buffer zone remaining above the spillway invert downstream of station 18+00.

Horizontal Buffer Zone Blasting

4.151 Instead of proposing an invert demonstration shot, the Contractor again wanted to utilize results of the 3 previous production shots, which were drilled on a variety of hole patterns. The Contractor felt production blasting techniques would be a suitable alternative to the time consuming, closely spaced hole pattern specified. The Government took the position that the Contractor's proposed alternative methods to demonstrate removal of the 4-foot horizontal buffer zone might not provide the control necessary to preclude overbreak and produce competent rock faces within the specified 9-inch tolerance limits at the final excavation lines. However, the Government agreed to evaluate the so-called "invert demonstration shots" provided a section within each blast area was thoroughly cleaned of all loose rock, and survey controls established so that the effectiveness of the excavation technique in maintaining grade control and controlling overbreak could be made.

4.152 Using a Case backhoe and laborers with shovels, an approximate 25 x 25-foot area between stations 16+75 and 17+00 near the spillway centerline was excavated and cleaned. A grade checker then measured elevations at various locations within the cleaned area. The subsequent evaluation confirmed the Government's position. The wide 8 x 8-foot production hole pattern and variations in hole depth and hole angle resulted in numerous high spots in blocky, less fractured andesite rock and localized overexcavated areas in highly fractured to shattered rock (photo 99). The detailed inspection indicated that a wide blast hole pattern would not produce a foundation surface which would fall within the 9-inch tolerance limits specified for the invert.

4.153 The other two hole patterns were not evaluated for the following reasons: (1) the 6 x 6-foot pattern was demonstrated in an area composed of flow breccia and tuff bedrock which is not representative of the bulk of the spillway excavation; and (2) since the 8 x 8-foot pattern produced unacceptable results, the wider 9 x 9-foot pattern would also be unacceptable.

4.154 The Contractor then proposed to remove the buffer zone using a 6 x 6-foot hole pattern with 1 foot of subdrilling. He felt a demonstration of his blasting methodology might impact the start of Stage III construction activities. The Government's response was that the Contractor would assume all responsibilities and costs for producing an acceptable invert surface within the tolerance limits specified. The remaining spillway excavation (exclusive of the sill) between stations 12+50 and 18+00 required two shots, blast nos. 15 and 16, on 17 September and 3 October 1984, respectively. A total of 10,868 cubic yards of rock was blasted using 7250 pounds of ANFO. This resulted in an average PF of 0.67. The blasting technique utilized was similar to the step-drilling plan utilized above the spillway bench. Trim line holes were not drilled but were offset 3 to 4 feet from the toe of the existing slope and drilled vertically due to Contractor concerns over possible ravelling and sloughing of the side slopes. Blast no. 16 was required to break up unexpected hard, resistant outcrops of tuff and flow breccia encountered during dozer excavation downstream of shot no. 15.

Rock Hauling

4.155 The muck generated by either ripping or blasting was removed from the spillway area using several methods. The type of method used was usually dependent on: (1) the availability of equipment, (2) site conditions, (3) end-product use of the excavated material, and (4) width of the cut. During the initial stages of mechanical excavation, the D9 dozer just pushed the muck over the steep front slope of the spillway in order to build up a ramp suitable for scraper traffic. Once an acceptable ramp was established, Caterpillar 651B scrapers began hauling rock to the miscellaneous fill area or to the Stage I diversion levee for use as slope protection (photo 100). Scraper use continued until the spillway excavation was down to bench grade. Rock from production blast no. 9, which was not wasted, was used for slope protection on the Stage II diversion levee and also for the Stage I embankment protective cover. The scrapers were very efficient in removing material but required a wider cut and free access in both directions to operate effectively. The spillway rock below the bench was removed using Terex rock trucks fed by a Caterpillar 992 front end loader (photo 101). This method proved the most suitable because: (1) access to the spillway was restricted to the upstream haul road only, (2) the increasingly narrower cut restricted equipment movement, and (3) it allowed removal of muck at a steep face created during "full-face" excavation of production shots from the upstream to the downstream end of the spillway. The shot-rock produced by blasting below the bench was either hauled to the miscellaneous fill area for disposal, or used in the downstream pervious shell zone of the dam embankment or as landscape stone.

4.156 In October 1984, the spillway excavation was completed down to invert grade and only the sill excavation and construction remained.

SPILLWAY SILL

4.157 The spillway is unlined except for a narrow concrete sill section at station 17+50. The main purpose of the sill is to provide a hydraulically controlled channel section for the PMF by protecting the integrity of the spillway crest invert elevation of 1456.2 feet. As originally designed, the

sill was to be trapezoidal in cross section, extending across the full width of the spillway floor and up both side slopes to the bench elevation. Although the maximum water surface for the PMF is 1481.1, the sill was extended up to the bench to avoid potential problems during excavation of the andesite bedrock (photo 102).

Excavation

4.158 Work on the sill section did not begin until late November 1984, as the Contractor concentrated on completing the dam embankment closure section. The Contractor did not drill and shoot the sill excavation as shown on plate 59 of the contract drawings. Instead he used a Gradall equipped with a hydraulic ram attachment to break up the rock, and a Case backhoe to excavate the broken rock (photo 103). A grade checker constantly monitored the progress of the excavation to insure that the sill was excavated as closely as possible to the specified lines and grades. Given the diverse geologic conditions, this method of excavation proved fairly successful in limiting severe overbreak in the andesite rock mass (photo 104). Deviations from neat line were unavoidable due to the irregularly spaced, blocky joint structures and, particularly on the side slopes, adversely oriented joint structures. The end result was an overall wider and somewhat deeper excavation which required approximately 2-1/2 times the amount of concrete estimated (photo 105).

Construction

4.159 The spillway sill was constructed as closely as possible to the original design configuration, although the 1V on 0.5H design side slopes were difficult to attain because of the irregularly shaped excavated slopes. Once the reinforcing steel was installed and the foundation surfaces cleaned of all loose material and wetted, 52 cubic yards of 3000 lb/in^2 structural concrete were placed in the bottom section of the sill excavation, and 35 cubic yards of pneumatically placed concrete (shotcrete) were used in lieu of poured-in-place concrete for the sidewalls. The constructed invert section does not lie flush with the surrounding rock as shown on the contract drawings, but instead projects an average of 6 inches to 1 foot above the existing grade due to variations within and outside the specified 9-inch tolerance limits for rock excavation. The concrete section varies from 7 to 8-1/2 feet in width across the bottom and is sloped down to bedrock on either side of station 17+50. The edges of the concrete were rounded to preclude cracking. Due to the non-uniformity of the excavated side slopes, the concrete sill sidewalls, instead of being flush with the surrounding rock are frequently recessed into the slope with rock projecting out on either side of the concrete sections (photo 106). The irregular shape of the sidewall excavations and the use of shotcrete instead of formed concrete resulted in a occasionally concave shaped side slopes varying in width from 4 to 10 feet.

SUMMARY AND CONCLUSIONS

4.160 According to the blasting summary in table 6, 85,298 cubic yards of material were excavated from the spillway using blasting techniques. This figure represents approximately 85 percent of the total spillway excavation. The Contractor used 64,264 pounds of explosives (mostly ANFO) to blast the

rock, resulting in an average PF of about 0.75. In the final analysis, the step-drilling and trim blast techniques employed produced acceptable results considering the diverse geologic conditions (photo 107). The variability in the excavation characteristics of the spillway rocks produced an end-product excavation which locally deviates from original lines and grades. Rockfalls which occurred during and after excavation have reduced bench widths from the specified 12 feet to as little as 6 feet in some areas (photo 108). These rockfalls may have been induced by blasting effects but more likely the presence of loosely keyed rock blocks and dip-slope joint planes precipitated slope failures. The side slopes appear to have stabilized, although minor ravelling and sloughing will probably continue to occur. This may require periodic maintenance cleaning of the benches and invert section.

Dike No. 1

DESCRIPTION

4.161 Dike no. 1 is a compacted, zoned earthfill structure composed of pervious shell zones, transition zones, and a central core zone. The upstream slope is protected by a 12-inch layer of Type I stone up to elevation 1475 and a 24-inch layer of Type I stone between elevation 1475 and the crest of the dike. The downstream slope is covered by 12 inches of Type III stone overlain by landscaping materials. The dike is required to control the SPF and PMF reservoir pools. The embankment plan, profile, and cross sections are shown on plates 53 through 56.

GEOLOGY

4.162 The foundation materials for dike no. 1 consist of Tertiary-age andesite bedrock and non-homogeneous alluvial soils. Generally two distinct soil layers are present: a maximum 2-foot thick sandy clay layer overlying a maximum 1-1/2-foot thick layer of caliche cemented sands which grades into a caliche cemented sandy gravel with rock fragments. This coarser layer extends down to a depth of at least 25 feet. The geologic conditions encountered north of dike station 78+65 were similar to those anticipated from the pre-construction geotechnical investigations except that a third soil layer previously recognized appears to be a coarser grained equivalent of the overlying thin caliche layer. South of station 78+65, no geotechnical investigations were conducted along the dike alinement. However, based on investigations conducted along an alternative alinement located 350 to 400 feet to the east, downstream subsurface conditions were anticipated to be essentially the same as those upstream and that bedrock would be below the invert depth of the exploration trench.

4.163 During construction, volcanic bedrock, classified petrographically as a porphyritic andesite was encountered at shallow depths below the caliche layer in the exploration trench excavation between stations 84+75 on the south abutment and station 78+65, see plate 53. The bedrock surface, extending from its surface contact above the south abutment north for a distance of approximately 610 feet, has the appearance of a gently dipping bedrock

pediment which probably drops off rather steeply upstream of station 78+65. The various lithologic units encountered are described separately followed by a discussion of the geologic structure.

Lithologic Units

4.164 The bottom of the exploration trench between stations 84+75 and 78+50 was mapped at a scale of 1 inch equals 10 feet. The foundation geology for dike 1 is presented on plates 43 and 44. What appeared to be two distinct rock types exposed in the exploration trench turned out to be variations of porphyritic andesite when samples were subjected to detailed petrographic analyses.

4.165 PORPHYRITIC ANDESITE. Most of the porphyritic andesite (Tvpa), is pinkish-gray in color, hard, moderately to highly fractured, and slightly weathered. South of station 82+25, the rock is generally a light pinkish-gray color, moderately hard, slightly to moderately weathered, with a less glassy matrix. Joint patterns also give the andesite in this reach a more platy appearance. These changes in rock characteristics may be due to this portion of the rock mass representing the upper weathered surface of the lava flow. The prinicipal constituents of the flow include fine grained, randomly dispersed phenocrysts of hornblende, biotite, magnetite, and feldspar in an aphanitic, glassy matrix. Examination of the feldspar crystals indicates the rock mass has undergone two periods of crystallization, one at depth and one during upward movement or emplacement. Scattered and locally concentrated zones containing abundant lenses and bands of a reddish-brown porphyritic andesite ranging from 1/2 inch to 12 inches in thickness are present. This rock contains fine to medium-grained phenocrysts of feldspar, hornblende, biotite, and magnetite randomly dispersed in an altered glassy matrix.

4.166 ALLUVIUM. Between stations 78+65 and 10+00, the dike embankment is founded on alluvium (Qoal), composed of well indurated, insoluble caliche with sand, gravel and rock fragments. This layer of hardpan appears to be quite extensive beneath the valley floor in the vicinity of the dike.

Geologic Structure

4.167 Although classified as an andesite, the bedrock, which outcrops intermittently on the northwest flank of the West Wing Mountains and which is exposed in the exploration trench, is texturally and structurally different from the aphanitic andesite on the right abutment and spillway. However, the exact field relationship between the two flows is unknown and sufficient petrographic data is not available to determine whether the rock units are genetically related. The porphyritic andesite was also assigned a Tertiary age because it was assumed that the two flows, although possibly resulting from separate volcanic episodes, are contemporaneous in terms of age.

4.168 Structurally, both andesite flows have recognizable blocky characters due to their probable highly viscous nature. The dike 1 block flow, however, does not exhibit the uniform strong angular jointing so noticeable in the abutment and spillway block flows but instead appears to have undergone locally extensive brecciation during emplacement (photo 109). Although well developed joint structures are present, there are numerous areas where

jointing is discontinuous and poorly developed, particularly in the brecciated zones. Coherent, large rock blocks are frequently surrounded by calichified andesite breccia while in other instances, the entire rock mass has been disrupted and subsequently calichified. The upper part of the andesite flow tends to have a platy more layered structure, with noticeable arcuate joints and more pronounced, thin (1/2 inch maximum) flow bands. Strong, subparallel to wedge-shaped linear joint structures are recognizable throughout the main body of the flow. A stereonet plot of joint attitudes measured in the exploration trench revealed 3 prominent joint systems: (1) striking N15°W, dipping 45°NE; (2) striking N5°W, dipping 90°; and (3) striking N60°W, dipping 60°SW. Upstream of station 83+50, there are also numerous joint planes with near-horizontal dips (averaging 15°NE). Joints are typically closely to moderately spaced (1 to 12 inches), with average apertures between 1/2 and 2 inches. Most joints are usually infilled to varying degrees with well indurated caliche or hard calichified breccia. The overall jumbled nature of the rock exposed in the trench excavation, which may represent the leading edge or margin of the flow, has tended to obscure any flow layering. Apparent layering, striking N60°W and dipping 30°NE, is present in the upper arched portion of the flow sheet, particularly around station 84+40. No faults were noted in the andesite flow.

EXCAVATION

4.169 Excavation for dike no. 1 began in December 1983 and was completed by January 1984. The excavation sequence consisted of stripping approximately the upper 1/2 to 2 feet of alluvium, followed by excavation of an average 5-foot deep exploration trench beneath the core zone. Stripping consisted of removing the first soil layer with a self-propelled elevating scraper down to the caliche layer or bedrock and then excavating 2 feet of caliche from beneath the upstream transition and pervious shell zones. The caliche was ripped with a D9 dozer and subsequently excavated using two push dozers and scrapers.

4.170 After stripping was completed, an exploration trench was excavated beneath the core zone (photo 110) to an average depth of 5 feet or sound bedrock as shown on plates 53 through 55. Only about 8 percent of the excavation was in bedrock with the remainder being in caliche (photo 111). The exploration trench was ripped with D9 dozers and then excavated with push dozers and scrapers. Removal of approximately 2 to 9 feet of andesite rock was required to reach a suitable foundation surface. An overall greater depth of excavation was required in the weathered portion of the andesite flow near the downstream end. The caliche or hardpan layer was so well indurated in some reaches that the dozer was unable to reach the designated depth and as a result hard humps were left in the bottom of the trench.

FOUNDATION PREPARATION AND TREATMENT

4.171 Only that portion of the exploration trench founded on bedrock, between stations 78+65 and 84+75, required formal foundation preparation and treatment. A backhoe equipped with scraper bar, shovels, picks, and a high pressure air hose were used to remove all loose and unsuitable materials from the bedrock surface and from open fractures during the initial cleaning of the trench bottom. Then the sides of the exploration trench were cleaned using shovels

and low pressure air. Only the more competent exposed bedrock and/or caliche was air blown to avoid removing excess material and creating overhangs and pockets not accessible to compaction equipment. Once the sides were cleaned, the trench bottom was hand picked of all remaining loose material and air blown again.

4.172 Following completion of foundation preparation, only minimal foundation treatment (i.e., dental concrete) was required because of the relatively smooth foundation surface and the general lack of extensive open fractures. Five cubic yards of low slump, 1000 lbs/in^2 minimum compressive strength concrete with 3/4-inch maximum size aggregate were placed in small, localized depressions and on highly fractured to shattered areas containing an abundance of open fractures. The rock areas designated to receive concrete were wetted immediately prior to placement and the concrete placed directly via the cement truck chute or by laborers using shovels. The concrete was then vibrated where possible to work it into place. Feather edges were avoided and the concrete surfaces shovel-tamped to facilitate better bonding with the core material. Dental concrete was used to protect intact rock areas susceptible to possible damage or degradation from compaction equipment and to provide a relatively smooth, level surface accessible to compaction equipment. Foundation grouting was not considered necessary for the bedrock portion of the exploration trench because most of the bedrock foundation was above the elevation of the SPF reservoir pool.

Dike No. 2

DESCRIPTION

4.173 Dike no. 2 is a compacted earthfill structure composed of pervious shell materials. The downstream slope is covered by 12 inches of Type III stone. The dike is required to control the PMF reservoir pool only. The embankment plan, profile, and typical cross section are shown on plate 57.

GEOLOGY

4.174 The foundation for dike no. 2 is composed of alluvium, generally silty, sandy gravel; and highly weathered granitic bedrock of the Precambrian basement complex. The foundation conditions were essentially the same as indicated on the contract drawings (plate 27).

EXCAVATION

4.175 Excavation consisted of stripping approximately the upper 6 inches to 1 foot of material with a D9H dozer (photo 112).

FOUNDATION PREPARATION

4.176 No formal preparation and treatment and geologic mapping of the foundation surface was required because the dike is composed only of pervious shell materials. After inspection and approval of the foundation by Corps personnel, the area was scarified to a depth of 6 inches, wetted, and proof rolled, prior to embankment material placement.

5. Embankment Placement

5.01 The following paragraphs briefly describe the equipment and procedures used to place and compact embankment materials at the rock contacts. A more detailed discussion of embankment construction, including the results of field and laboratory testing, is presented in the New River Dam Embankment Performance and Criteria Report (U.S. Army Corps of Engineers, in prep.)

Core Materials

5.02 Special procedures were used in placing and compacting core materials at the rock contacts within the core zones of the dam and dike no. 1 embankments in order to insure a suitable embankment/foundation bond. The initial lifts of wet-of-optimum core material were placed in 6 to 12-inch thicknesses on the cleaned and wetted bedrock surface with a rubber tired front end loader. Placement of wetter core materials was done not only to insure its bonding to the rock but to maximize the filling of voids and cracks in the rock with core materials. Compaction was accomplished by 8-wheel passes of the front end loader with a loaded bucket (photos 113 and 114). Wheel rolling was used to prevent damage to the treated bedrock surface by the tamping roller (photo 115). The compacted surface was scarified by back dragging the bucket teeth prior to placing the next lift. Compaction with a tamping roller was initiated when a sufficient thickness of material covered the bedrock surface. Core materials were placed and compacted on a 4H to 1V slope against each abutment (photo 116). Establishment of these ramps allowed the tamping roller to eventually compact closer to the abutment since the rock surface was protected from the tamping roller by a layer of core material. The same placement and compaction procedures were used against the 1V on 1H slopes of the concrete plug within the dam embankment core zone.

5.03 The foundation depression in the Stage II core trench was backfilled with core materials (photo 26). The bottom and slopes of the depression were prepared using brooms, shovels, buckets and low pressure air blasting. Once all loose material was removed, the cleaned bedrock was wetted prior to the placement of core materials. The core materials were placed wet of optimum in 4-inch loose lifts and compacted with hand held power tampers. Each lift was then scarified before the next lift was placed. Similar materials and procedures were employed in the backfilling of the backhoe trench between approximately stations 29+55 and 30+20 in the Stage I excavation.

Transition Materials

5.04 No special procedures were used in placing and compacting transition materials at the rock contacts. The materials were spread in 12-inch lifts by a motor grader or dozer. Each lift was compacted by four passes of a steel drum vibratory roller. Nested cobbles at the rock contacts were removed prior to compaction of the lift. The compacted surface of the preceding lift was scarified to a depth of 6 inches with the rippers on a motor grader prior to placement of the next lift.

Pervious Shell Materials

5.05 No special procedures were used in placing and compacting pervious shell materials at the rock contacts. The materials were spread in 24-inch lifts by a motor grader or dozer. Each lift was compacted by four passes of a steel drum vibratory roller. Nested cobbles at the rock contacts were removed prior to compaction of the lift. The compacted surface of the preceding lift was scarified to a depth of 6 inches with the rippers on a motor grader prior to placement of the next lift.

6. Possible Future Problems

6.01 No significant geologic features or foundation conditions were encountered which might affect the integrity of the dam embankment and its appurtenances. Two minor, though potentially recurrent, problems which might require future monitoring are discussed in the following paragraphs.

Outlet Channel Erosion

6.02 During the first periodic inspection of New River Dam on 21 March 1985, erosion was noted in two areas of the outlet channel. Low flows through the outlet channel had removed an approximately 2 to 3-foot section of the 9-inch layer of Type III stone from the lower portion of the 1V on 2.5H side slopes between stations 5+06 and 6+00 and had eroded a near vertical cut in the exposed alluvial foundation materials. There was also some undercutting of the 24-inch grouted stone section on the east slope at station 5+06. At the downstream end of the outlet channel, low flows had also scoured out a depression in the compacted backfill placed against the grouted stone cutoff. By the time of the final inspection on 19 April 1985, these two problem areas had been repaired. The erosion of the outlet channel side slopes was repaired by the placement of a thicker section of Type III stone. The depression at the end of the grouted stone section in the streambed was backfilled with Type III stone. According to the original design, Type III stone protection was not necessary for the lower portion of the outlet channel bottom and side slopes; it was placed only to provide extra protection. Despite the presence of Type III stone, erosion still occurred. However, erosion within the channel will not compromise the stability of the dam but should be monitored if future periodic maintenance of the channel is necessary.

Spillway Slope Degradation

6.03 During the two post construction inspections mentioned in the preceeding paragraph, little change was noted in the condition of the spillway side slopes and benches since the completion of excavation (see para. 4.160). The side slopes appeared to have stabilized although minor ravelling and sloughing can be expected to continue because of the highly fractured nature of the andesite rock mass. Therefore, periodic maintenance cleaning of the benches and invert section may be required. Areas subject to potentially major rockfalls will be those sections of the andesite rock mass where the joint planes have adverse orientations toward the excavation. Another possible problem area is in the vicinity of station 14+00, north side, where a significant slope failure occurred due to the intensely fractured, platy nature of the bedrock (see pl. 42 and photo 117). Transverse cracks, approximately 1 to 2 feet back from the upper edge of the excavation in this area, indicate the potential for recurring rockslides. Major rockfalls would not reduce the effectiveness of the spillway.

6.04 As discussed previously in this report, excavated andesite rock has the potential to undergo extensive breakdown. The responsible mechanism for this breakdown is still in question. This breakdown phenomenon was first noted during a 1982 inspection of backfilled preconstruction spillway test trenches excavated approximately 1 year earlier. An examination of the spillway excavation during the first periodic inspection indicated that breakdown of talus on the bench and upper slope on the north side downstream of station 15+00 had begun to occur. This corresponds very closely with the location of test trench TT-94 (see pl. 18) where the most severe degradation was noted. During the subsequent final inspection, it was apparent that some of the loose in-place rock in this same area was now beginning to breakdown. Although field evidence suggests that the rock breakdown tends to be limited to the downstream portion of the spillway, it is not known whether other areas will be similarly affected. During future periodic inspections, the condition of the spillway side slopes and benches, particularly in those areas where rock breakdown is occurring, should be monitored closely and any potential impact on structures like the concrete sill, and fence posts and guard rails near the edges of the excavation should be noted.

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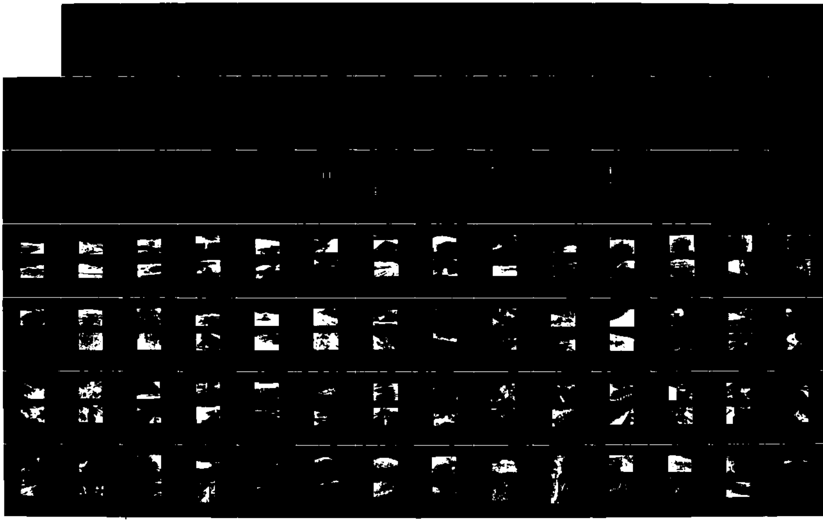

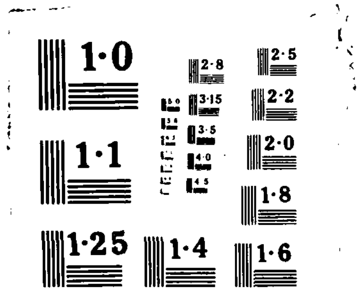
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TABLE 1. CONTRACT QUANTITIES

Item No.	Description	Estimated Quantity	Unit	Unit Price	Actual Quantity
1.	MOBILIZATION AND PREPARATION WORK	1	Job	L.S.	1
2.	DIVERSION AND CONTROL OF WATER	1	Job	L.S.	1
3.	CLEARING AND GRUBBING	1	Job	L.S.	1
4.	STRIPPING, DIKES 1 & 2	85,000	Cu. Yd.	$1.25	68,540
5.	EXCAVATION, DIKE NO. 1 (EXPLORATION TRENCH)	33,000	Cu. Yd.	$1.75	41,443
6.	SCALING				
	(a) FIRST 1000 Cu. Yd.	1,000	Cu. Yd.	$17.00	0
	(b) OVER 1000 Cu. Yd.	500	Cu. Yd.	$16.00	0
7.	EXCAVATION				
	DAM EAST ABUTMENT	11,700	Cu. Yd.	$7.00	8,313
	DAM WEST ABUTMENT	2,000	Cu. Yd.	$16.00	8,653
8.	EXCAVATION, FOUNDATION (DAM)	325,000	Cu. Yd.	$1.25	288,106
9.	EXCAVATION, CORE-TRENCH (DAM)	60,000	Cu. Yd.	$1.60	77,360
10.	FOUNDATION PREPARATION				
	(a) FIRST 1,500 Man Hr.	1,500	Man Hr.	$30.00	1,500
	(b) OVER 1,500 Man Hr.	1,500	Man Hr.	$28.00	5,813
11.	EXCAVATION, OUTLET WORKS	29,000	Cu. Yd.	$ 7.00	41,652
12.	EXCAVATION, SPILLWAY	96,000	Cu. Yd.	$ 5.00	101,181
13.	EXCAVATION, ACCESS ROAD	5,500	Cu. Yd.	$ 4.00	6,052
14.	EXCAVATION, TOE	25,000	Cu. Yd.	$ 1.25	10,000
15.	FILL, OUTLET WORKS	15,000	Cu. Yd.	$ 2.50	17,094
16.	FILL, ACCESS ROAD	71,000	Cu. Yd.	$ 1.50	73,676
17.	FILL, CORE	524,000	Cu. Yd.	$ 1.50	539,214
18.	FILL, TRANSITION	410,000	Cu. Yd.	$ 1.75	422,265
19.	FILL, PERVIOUS SHELL	1,640,000	Cu. Yd.	$ 1.50	1,627,807

TABLE 1. (Continued)

Item No.	Description	Estimated Quantity	Unit	Unit Price	Actual Quantity
20.	FILL, TOE	21,000	Cu. Yd.	$ 1.25	15,884
21.	FILL, MISCELLANEOUS	245,000	Cu. Yd.	$ .40	154,684
22.	ADDITIONAL ROLLING	500	Hours	$90.00	0
23.	GRAVEL DRAIN	9,000	Cu. Yd.	$ 7.00	9,000
24.	STONE, TYPE I	35,000	Ton	$ 5.00	35,000
25.	STONE, TYPE II	40,000	Ton	$ 5.00	40,000
26.	STONE, TYPE III	50,000	Ton	$6.00	50,000
27.	GROUTING, STONEWORK	1,600	Cu. Yd.	$50.00	1,600
28.	CONCRETE, CONDUIT	1,400	Cu. Yd.	200.00	1,400
29.	CONCRETE, OUTLET CHANNEL SILL	13	Cu. Yd.	$100.00	13
30.	CONCRETE, SPILLWAY SILL	36	Cu. Yd.	$125.00	36
31.	INTAKE STRUCTURE	1	Job	L.S.	1
32.	ENERGY DISSIPATOR	1	Job	L.S.	1
33.	CONCRETE PLUG (LEAN MIX)				
	(a) FIRST 300 Cu. Yd.	300	Cu. Yd.	$50.00	300
	(b) OVER 300 Cu. Yd.	100	Cu. Yd.	$45.00	100
34.	CONCRETE, DENTAL				
	(a) FIRST 1000 Cu. Ft.	1,000	Cu. Yd.	$90.00	1,000
	(b) OVER 1000 Cu. Ft.	750	Cu. Yd.	$85.00	750
35.	GROUT, SLURRY				
	(a) FIRST 500 Cu. Ft.	500	Cu. Ft.	$22.00	500
	(b) OVER 500 Cu. Ft.	500	Cu. Ft.	$20.00	500
	OPTION NO. 1				
36a.	PORTLAND CEMENT	8,400	Cwt.	$7.00	8,400
37.	STEEL REINFORCEMENT	240	Ton	$700.00	240
38.	WATERSTOP	400	Lin. Ft.	$9.00	400

TABLE 1. (Continued)

Item No.	Description	Estimated Quantity	Unit	Unit Price	Actual Quantity
39.	FOUNDATION DRILLING AND GROUTING				
	(a) MOBILIZATION AND DEMOBILIZATION	1	Job	L.S.	1
	(b) DRILLING EXPLORATORY GROUT HOLES	350	Lin. Ft.	$25.00	350
	(c) DRILLING GROUT HOLES	9,100	Lin. Ft.	$8.00	9,100
	(d) PIPE FOR GROUT HOLES	500	Lin. Ft.	$5.00	500
	(e) DRILL SET-UPS				
	(1) GROUT HOLES	370	Each	$25.00	370
	(2) EXPLORATORY GROUT HOLES	4	Each	$50.00	5
	(f) PRESSURE TESTING	175	Hour	$60.00	47.5
	(g) GROUT PUMP CONNECTIONS	375	Each	$50.00	225
	(h) PLACING GROUT	2,500	Sack	$20.00	2,097.5
40.	PIPE GATE, OUTLET WORKS	1	Each	$1,500.00	6
41.	DRIVE GATE	1	Each	$1,000.00	1
42.	DOUBLE DRIVE GATE	2	Each	$1,500.00	5
43.	REINFORCED CONCRETE CULVERT, DIKE NO. 1	1	Job	L.S.	1
44.	CORRUGATED METAL PIPE, 24 INCH	60	Lin. Ft.	$35.00	60
45.	CORRUGATED METAL PIPE, 36 INCH	60	Lin. Ft.	$55.00	60
46.	CORRUGATED METAL PIPE, 48 INCH	660	Lin. ft.	$65.00	660
47.	METAL END SECTIONS FOR 36" CMP	2	Each	$300.00	2
48.	METAL END SECTIONS FOR 24" CMP	2	Each	$200.00	2
49.	AGGREGATE BASE, ROAD	2,400	Cu. Yd.	$9.00	2,353
50.	ASPHALT CONCRETE PAVEMENT	2,800	Ton	$35.00	1,039

TABLE 1. (Continued)

Item No.	Description	Estimated Quantity	Unit	Unit Price	Actual Quantity
51.	5' CHAIN LINK FENCE	16,000	Lin. Ft.	$8.00	16,512
52.	GUTTER	4,800	Lin. Ft.	$15.00	7,992
53.	GUARDRAIL	450	Lin. Ft.	$15.00	410
54.	LOG BARRIER	38	Lin. Ft.	$25.00	40
55.	PROJECT SIGN	1	Job	L.S.	1
56.	HYDROLOGIC FACILITIES	1	Job	L.S.	1
57.	STAFF GAGES AND MONUMENT	1	Job	L.S.	1
58.	GAGING STATION BRIDGE	1	Job	L.S.	1
59.	TOPSOILING	23,000	Cu. Yd.	$300.00	22,369
60.	LANDSCAPE STONE	11,300	Cu. Yd.	$6.00	16,283
61.	DESERT GRAVEL	4,100	Cu. Yd.	$15.00	4,367
62.	DESERT VARNISH FINISH	4,500	Gal.	$20.00	4,500
63.	CONCRETE STAIN	330	Gal.	$25.00	330
64.	SEEDING	50	Acre	$2,500.00	50
65.	PLANTING	1	Job	L.S.	1

TABLE 2. GEOTECHNICAL RELATED CONTRACT MODIFICATIONS

Mod. No.	Item	Description of Change	Cost
P00005	Stage 1 Excavation	Excavation of core trench from elevation 1365 as shown on the contract drawings to elevation 1355, from Station 29+64 to Station 31+90.	$ 68,227.00
P00009	Grouted Stone Upstream Intake Structure	Place and grout stone upstream of Intake Structure, Station 20+99.27 to Station 21+30.	$ 24,779.00
P00010	Stone Protection	Contractor was required to significantly alter borrow processing methods and procedures in order to obtain adequate and acceptable Type I and II Stone.	$480,000.00
P00013	Observation Wells	Install three observation wells at the New River Dam Site.	$ 53,877.00

TABLE 3. OBSERVATION WELL NO. 1

Section: 35
Township: 5N
Range: 1E
Location: 350 ft. downstream of dam station 24+50

Date drilled: February 1985
Total depth of well: 144 ft.
Elevation, top of pipe: 1386.8 ft.
Standing water level: 36 ft.
Total amount of casing: 115 ft.
Top of perforations: 25 ft.
Bottom of perforations: 110 ft.
Amount of PVC pipe installed: 136 ft.

Driller's Log:

0 to 10 ft.	Silty sand and boulders
10 to 57 ft.	Sand, gravel and boulders
57 to 144 ft.	Sand, clay and gravel
144 to 147 ft.	Hard rock

Standing water level: 38 ft. (Sept. 1985)
Taped well depth: 131 ft. (Sept. 1985)

TABLE 4. OBSERVATION WELL NO. 2

Section: 35
Township: 5N
Range: 1E

Location: 350 ft. upstream of dam station 24+50

Date drilled:	February-March 1985
Total depth of well:	138 ft.
Elevation, top of pipe:	1399.7 ft.
Standing water level:	43 ft.
Total amount of casing:	136.6 ft.
Top of perforations:	25 ft.
Bottom of perforations:	130 ft.
Amount of PVC pipe installed:	136 ft.

Driller's Log:

0 to 20 ft.	Silty sand and boulders
20 to 32 ft.	Sand and gravel
32 t0 47 ft.	Loose sand and gravel
47 to 95 ft.	Sand, clay and gravel
95 to 113 ft.	Sand, clay and loose rock
113 to 134 ft.	Sand, clay and rock
134 to 138 ft.	Hard rock

Standing water level:	45.5 ft. (Sept. 1985)
Tape well depth:	128.5 ft. (Sept. 1985)

TABLE 5. OBSERVATION WELL NO. 3

Section: 23
Township: 5N
Range: 1E
Location: Approx. 2-1/4 miles upstream of dam

Date drilled: March-April 1985
Total depth of well: 155 ft.
Elevation, top of pipe: 1463.0 ft.
Standing water level: 121 ft.
Total amount of casing: 146.3 ft.
Top of perforations: 25 ft.
Bottom of perforations: 140 ft.
Amount of PVC pipe installed: 150 ft.

Drillers Log:

0 to 36 ft.	Silty sand and rock
36 to 42 ft.	Heaving sand and rocks
42 to 48 ft.	Sand and hard rock
48 to 49 ft.	Hard rock
49 to 53 ft.	Clay and hard rock
53 to 145 ft.	Sandy clay and gravel
145 to 155 ft.	Hard rock

Standing water level:	132.5 ft. (Sept. 1985)
Taped well depth:	150 ft. (Sept. 1985)

TABLE 6. NEW RIVER DAM, ARIZONA - BLASTING SUMMARY

BLAST NO.	DATE BLASTED	STATIONING	TOP ELEVATION	DEPTH DRILLED (FT)	BOTTOM ELEVATION	PRODUCTION HOLE DIA. (IN.)	PATTERN (FT.)	STEMMING (FT.)	POUNDS EXPLOSIVES	NUMBER OF DELAYS	MAXIMUM POUNDS/DELAY	CUBIC YARDS ROCK IN PLACE	POWER FACTOR	PRODUCTION HOLES	SLOPE-LINE HOLES	REMARKS
		Right Abutment														
1 2 3	11/8/83 11/11/83 11/15/83	? ? ?	? ? ?	5-7 5-7 8-10	? ? ?	1½ 1½ 1½	4x5 4x5 4x5	3 3 3	234 72 124	12 12 8	19½ 6 16	266 80 206	.87 .90 .60	183 106 31		blasting for abutment access road and staging platform
		Spillway														
4	11/18/83	18+70- 18+82	1474 1471	10 18	1464 1453	2½	6x7	4	183	2	120	279	.66	13	12	20 ft. pre-splits drilled on ½:1 on 3 ft. centers (unacceptable presplit demo.)
4a	11/13/83	17+25- 17+43	1511 1511 1511	6 12 18	1505 1499 1493	2½	6x7	4 5 7	164	3	80	240	.68	12	7	20 ft. presplits drilled on 3/4:1 on 3 ft. centers (unacceptable presplit demo.)
8	2/9/84	17+00- 17+48	1503 1503 1503	7 15 24	1496 1488 1479	2½	8x8	3 5 5	1317	6	291	1621	.81	36		acceptable "step drilling" demo. for upper 1:1 side slopes

TABLE 6. NEW RIVER DAM, ARIZONA - BLASTING SUMMARY

BLAST NO.	DATE BLASTED	STATIONING	TOP ELEVATION	DEPTH DRILLED (FT)	BOTTOM ELEVATION	PRODUCTION HOLE DIA. (IN.)	PATTERN (FT.)	STEMMING (FT.)	POUNDS EXPLOSIVES	NUMBER OF DELAYS	MAXIMUM POUNDS/DELAY	CUBIC YARDS ROCK IN PLACE	POWER FACTOR	PRODUCTION HOLES	SLOPE-LINE HOLES	REMARKS
9	3/8/84	15+00- 18+00	1486- 1504	6- 20	1480- 1484	2½	8x8	4 ave	21550	19	1200	27217	.79	638		production shot to bench elev.
10	6/19/84	16+75- 17+20	1482- 1484	15 23	1400- 1461	3	8x8	3 5	1650	6	300	1848	.86	36	15	23 ft. presplits drilled on ½:1 slope on 3 ft. centers (unacceptable presplit demo.)
11	6/27/84	15+77- 16+25	1482	11 23	1458- 1459	3	6x8 8x8	5 5	1550	6	258	1798	.86	42	13	23 ft. trim-line holes drilled on ½:1 slope on 4 ft. centers (acceptable controlled blasting technique)
12	7/16/84	18+00- 19+10	1482- 1458	6- 30	1452	3	8x8	8 ave	7000	11	636	9385	.75	165	58?	unapproved production shot to invert elevation
13	8/21/84	16+25- 18+00	1483- 1484	22 ave	1461- 1462	3	9x9	4 ave	8200	10	820	11946	.69	180	?	production shot above horizontal buffer zone

TABLE 6. NEW RIVER DAM, ARIZONA - BLASTING SUMMARY

BLAST NO.	DATE BLASTED	STATIONING	TOP ELEVATION	DEPTH DRILLED (FT)	BOTTOM ELEVATION	PRODUCTION HOLE DIA. (IN.)	PATTERN (FT.)	STEMMING (FT.)	POUNDS EXPLOSIVES	NUMBER OF DELAYS	MAXIMUM POUNDS/DELAY	CUBIC YARDS ROCK IN PLACE	POWER FACTOR	PRODUCTION HOLES	SLOPE-LINE HOLES	REMARKS
14	8/27/84	13+25-14+00 14+00-16+25	1464-1483	14 ave 22 ave	1450-1461	3	6x6 9x9	4 ave.	15400	12	1284	20096	.76	419	150	production shot above buffer zone
15	9/17/84	13+75?-18+00	1455-1464	8 ave.	1447-1456	3	6x6	5 ave.	6700	13	773	10001	.69	940		invert buffer zone production shot
16	10/3/84	12+50-13+35?	1448-1450	4 ave.	1444-1446	3	6x6	3½ ave.	550	5	110	867	.63	163		production shot to remove hard rock humps above invert
		Outlet Works														
5	1/24/84	16+36-15+16	1386	1½-20	1385-1369	2½	5x6 6x6 6x7	6 ave.	1700	15	170	2589	.65	177		outlet works blasting conducted without benefit of demonstration
6	2/3/84	15+04-14+20	1383-1385	4-18	1369-1376	2½	5x6 6x6 6x7	5½ ave.	1500	19	86	2268	.66	157		
7	2/7/84	14+13-13+64	1385-1382	7	1376	2½	7x7	3	187	5	43	328	.57	26		

TABLE 7. NEW RIVER DAM, ARIZONA — FOUNDATION GROUTING

		RIGHT (WEST) SIDE															
		ABUTMENT (STA. 31+90 TO STA. 33+27)					CORE TRENCH (STA. 29+64 TO STA. 31+90)					COMBINED (STA. 29+64 TO STA. 33+27)					TOTALS
		P	S	T	Q_1	Q_2	P	S	T	Q_1	Q_2	P	S	T	Q_1	Q_2	
ZONE I	LF	450	175				575					1025	175				1200
	SACKS	81.0	13.0	N/A	N/A	N/A	2.7	N/A	N/A	N/A	N/A	83.7	13.0	N/A	N/A	N/A	96.7
	SACKS/LF	0.2	0.07				<0.01					0.08	0.07				0.08
ZONE II	LF	250	25				575	150	100			825	175	100			1100
	SACKS	89.5	0.0	N/A	N/A	N/A	98.1	51.6	0.0	N/A	N/A	187.6	51.6	0.0	N/A	N/A	239.2
	SACKS/LF	0.4	0.0				0.2	0.3	0.0			0.2	0.3	0.0			0.2
ZONE III	LF	50	25				300	275	150	25		350	300	150	25		825
	SACKS	50.6	5.9	N/A	N/A	N/A	467.3	32.4	51.6	0.0	N/A	517.9	38.3	51.6	0.0	N/A	607.8
	SACKS/LF	1.0	0.2				1.6	0.1	0.3	0.0		1.5	0.1	0.3	0.0		0.7
ALL ZONES	LF	750	225				1450	425	250	25		2200	650	250	25		3125
	SACKS	221.1	18.9	N/A	N/A	N/A	568.1	84.0	51.6	0.0	N/A	789.2	102.9	51.6	0.0	N/A	943.7
	SACKS/LF	0.3	0.08				0.4	0.2	0.2	0.0		0.4	0.2	0.2	0.0		0.3

SEE SHEET 3 FOR EXPLANATORY NOTES.

TABLE 7. NEW RIVER DAM, ARIZONA – FOUNDATION GROUTING

		LEFT (EAST) SIDE															
		CORE TRENCH (STA. 13+00 TO STA. 21+06)					ABUTMENT (STA. 10+00 TO STA. 13+00)					COMBINED (STA. 10+00 TO STA. 21+06)					TOTALS
		P	S	T	Q_1	Q_2	P	S	T	Q_1	Q_2	P	S	T	Q_1	Q_2	
ZONE I	LF	1960	150	50	50		750	50				2710	200	50	50		3010
	SACKS	99.8	38.8	39.8	2.0	N/A	23.0	5.1	N/A	N/A	N/A	122.8	43.9	39.8	2.0	N/A	208.5
	SACKS/LF	0.05	0.3	0.8	0.04		0.03	0.1				0.05	0.2	0.8	0.04		0.07
ZONE II	LF	1975	125	75	100	50	525	125	100	50	50	2500	250	175	150	100	3175
	SACKS	121.3	42.3	77.6	13.5	13.2	100.5	43.3	31.7	44.6	5.8	221.8	85.6	109.3	58.1	19.0	493.8
	SACKS/LF	0.06	0.3	1.0	0.1	0.3	0.2	0.3	0.3	0.9	0.1	0.09	0.3	0.6	0.4	0.2	0.2
ZONE III	LF	50										50					50
	SACKS	0.0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	0.0	N/A	N/A	N/A	N/A	0.0
	SACKS/LF	0.0										0.0					0.0
ALL ZONES	LF	3985	275	125	150	50	1275	175	100	50	50	5260	450	225	200	100	6235
	SACKS	221.1	81.1	117.4	15.5	13.2	123.5	48.4	31.7	44.6	5.8	344.6	129.5	149.1	60.1	19.0	702.3
	SACKS/LF	0.06	0.3	0.9	0.1	0.3	0.1	0.3	0.3	0.9	0.1	0.07	0.3	0.7	0.3	0.2	0.1
														GRAND TOTALS BOTH SIDES		LF	9360
																SACKS	1646
																SACKS/LF	0.2

SEE SHEET 3 FOR EXPLANATORY NOTES.

TABLE 7. FOUNDATION GROUTING

(notes)

1. Grout hole series are designated by P, S, T, Q_1 and Q_2.
P = primary grout hole; S = secondary grout hole; T = tertiary grout hole; Q_1 = quaternary grout hole; Q_2 = quinary grout hole.

2. Grout hole drilling footage per zone is expressed in linear feet (LF).

3. Grout take per zone is expressed in sacks of cement placed.

4. Drilling footage included only for those zones where split-spacing actually required due to high grout takes in adjacent grout holes. Exploratory grout hole footage not included.

5. Sack figures do not include backfilling or exploratory grout hole takes.

6. For Zone III on the west side, primary hole spacing was based on 20 foot centers. The following primary holes were considered secondary holes during Zone III grouting due to 20 ft. primary hole spacing: 30+10, 30+30, 30+50, 30+70, 30+90, 31+10, 31+30, 31+50, 31+70, 31+90, 32+00, (11 holes, 275 feet). The following secondary holes were considered tertiary holes due to 20 ft. primary hole spacing: 30+15, 30+65, 30+75 (3 holes, 75 feet).

TABLE 8. NEW RIVER DAM, ARIZONA – FOUNDATION GROUTING SUMMARY

HOLE NO.	SERIES	HOLE LOCATION: DAM STATION	HOLE LOCATION: OFFSET	G.S. ELEV. (ft.)	INCLINATION/ BEARING	DEPTH (ft.)	PRESSURE TESTING: FLOW (gpm)	PRESSURE TESTING: PRESS. (psi)	PRESSURE TESTING: K VALUE (ft./day)	DATE GROUTED	GROUTING: TAKE (sacks)	GROUTING: PRESS. (psi)	GROUTING: MIX (W/C)	REMARKS
10+10	P	10+09.5	0.5R	1474.2	30E	0-25	0.0	10	0.0	d				
10+20	P	10+20	0	1471.8	30E	0-25	0.0	10	0.0	d				
10+30	P	10+29.3	0	1469.1	30E	0-25	0.0	10	0.0	d				
10+40	P	10+40	0	1465.5	30E	0-25	0.0	10	0.0	d				
10+50	P	10+49.5	0	1461.5	30E	0-25	0.0	10	0.0	d				
10+60	P	10+60	0	1457.2	30E	0-25	1.2	10	0.2	2/9/84	0.3	10	6:1	
10+70	P	10+70	0	1455.1	30E	0-25	0.0	10	0.0	d				
10+80	P	10+79.3	0	1450.1	30E	0-25	0.0	10	0.0	d				
10+90	P	10+90	2.2L	1446.3	30E	0-25	0.2	10	0.04	d				
11+00	P	10+95.5	0	1443.4	30E	0-25	0.9	10	0.2	2/2/84	0.4	10	6:1	
						25-50	1.5	15	0.2	2/7/84	0.9	15	6:1	
11+10	P	11+10	0	1439.8	30E	0-25	1.2	10	0.2	2/2/84	0.4	10	6:1	
						25-50	5.5	15	0.5	2/7/84	1.7	15	6:1	
11+20	P	11+20	0	1436.4	30E	0-25	5.3	10	1.1	2/2/84	0.9	10	6:1	
						25-50	5.4	15	0.5	2/7/84	0.8	15	6:1	
11+30	P	11+30.5	0	1434.1	30E	0-25	0.9	10	0.2	2/2/84	0.2	10	6:1	
						25-50	2.2	15	0.2	2/7/84	0.5	15	6:1	
11+40	P	11+40	0	1430.5	30E	0-25	1.2	10	0.2	2/2/84	0.5	10	6:1	
						25-50	7.0	15	0.7	2/7/84	0.8	15	6:1	
11+50	P	11+50	0	1424.4	30E	0-25	2.2	10	0.5	2/2/84	1.5	10	6:1	
						25-50	0.0	15	0.0	d				
11+60	P	11+60	0	1421.8	30E	0-25	2.4	10	0.5	2/1-2/2 84	1.2	10	6:1	placed 0.1 sacks on 2/1/84 prior to equipment breakdown
						25-50	5.5	15	0.5	2/7/84	0.7	15	6:1	
11+70	P	11+70	0	1420.0	30E	0-25	5.4	10	1.1	2/1/84	1.5	10	6:1	
						25-50	6.3	15	0.6	2/7/84	0.7	15	6:1	
11+80	P	11+80	0	1417.2	30E	0-25	3.8	10	0.8	2/1/84	0.6	10	6:1	
						25-50	7.7	15	0.8	2/7/84	0.8	15	6:1	
11+90	P	11+90	0	1414.5	30E	0-25	0.3	10	0.06	2/1/84	0.3	10	6:1	
						25-50	11.3	15	1.1	2/7/84	0.7	15	6:1	

SEE SHEETS 14 & 15 FOR EXPLANATORY NOTES.

TABLE 8. NEW RIVER DAM, ARIZONA – FOUNDATION GROUTING SUMMARY

HOLE NO.	SERIES	HOLE LOCATION		G.S ELEV. (ft.)	INCLINATION/ BEARING	DEPTH (ft.)	PRESSURE TESTING			DATE GROUTED	GROUTING			REMARKS
		DAM STATION	OFFSET				FLOW (gpm)	PRESS (psi)	K VALUE (ft./day)		TAKE (sacks)	PRESS (psi)	MIX (W/C)	
11+95	S	11+96.3	0	1412.3	30E	0-50	7.0	15	0.7	2/9/84	4.3	15	6:1	
12+00	P	12+00.5	0	1409.9	30E	0-25	0.0	10	0.0	d				
						25-50	14.4	15	1.4	2/6/84	13.2	15	6/5:1	
12+05	S	12+05	0	1408.9	30E	0-50	9.2	15	0.9	2/9/84	2.2	15	6:1	
12+10	P	12+10.5	0.4L	1409.2	30E	0-25	1.8	10	0.4	2/1/84	0.8	10	6:1	
						25-50	0.5	15	0.09	d				
12+20	P	12+20	1.5L	1408.1	30E	0-25	0.0	10	0.0	d				
						25-50	0.9	15	0.09	d				
12+30	P	12+30	1.2L	1402.1	30E	0-25	1.4	10	0.3	1/25/84	0.4	10	6:1	
						25-50	1.1	15	0.1	2/6/84	0.7	15	6:1	
12+40	P	12+40	1.2L	1401.3	30E	0-25	0.0	10	0.0	d				
						25-50	0.7	15	0.07	d				
12+50	P	12+50	1.0L	1400.7	30E	0-25	0.9	10	0.2	1/25/84	0.4	10	6:1	
						25-50	6.4	15	0.6	2/6/84	0.8	15	6:1	
12+60	P	12+59.5	1.2L	1398.0	30E	0-25	2.3	10	0.5	1/24/84	1.0	10	6:1	
						25-50	0.9	15	0.09	2/6/84	0.6	15	6:1	
12+65	S	12+65	1.5L	1397.6	30E	0-25	8.3	10	1.7	1/31/84	3.2	10	6:1	
12+70	P	12+70	1.0L	1397.9	30E	0-25	7.3	10	1.5	1/24/84	7.1	10	6/4:1	½ hr circulation due to outlet works blast may have caused grout to set-up; decided to split-space
						25-50	6.3	15	0.6	2/6/84	0.9	15	6:1	
12+75	S	12+75	0	1397.6	30E	0-25	5.2	10	1.1	1/31/84	1.9	10	6:1	
102C	E	12+78.5	1.5L	1397.3	vert.	0-41.9	13.2	10	-	2/16/84	57.1	10-20	6/1:1	lost circulation during drilling at 38.3 feet
12+80	P	12+80.5	0.5L	1396.8	30E	0-25	1.5	10	0.3	1/24/84	1.4	10	6:1	
						25-50	7.1	15	0.7	2/6/84	3.0	15	6:1	
12+85	S	12+84	0.8L	1396.4	30E	0-50	5.8	15	0.6	2/6/84	5.3	15	6:1	
12+87.5	T	12+89	0	1395.5	30E	0-50	1.2	14	0.1	2/24/84	0.4	14	6:1	

SEE SHEETS 14 & 15 FOR EXPLANATORY NOTES.

TABLE 8. NEW RIVER DAM, ARIZONA – FOUNDATION GROUTING SUMMARY

HOLE NO.	SERIES	HOLE LOCATION: DAM STATION	HOLE LOCATION: OFFSET	G.S. ELEV (ft.)	INCLINATION/ BEARING	DEPTH (ft.)	PRESSURE TESTING: FLOW (gpm)	PRESSURE TESTING: PRESS. (psi)	PRESSURE TESTING: K VALUE (ft./day)	DATE GROUTED	GROUTING: TAKE (sacks)	GROUTING: PRESS. (psi)	GROUTING: MIX (W/C)	REMARKS
12+90	P	12+90	1.0L	1394.8	30E	0-25	7.4	10	1.5	1/24/84	3.7	10	6:1	
						25-50	8.4	15	0.8	1/30/84	14.4	15	6/4:1	
12+90.6	Q_2	12+90.2	2.5L	1395.3	30E	0-50	10.5	15	1.0	2/13/84	2.5	15	6:1	grout leak at surface from hole 12+91.9
12+91.3	Q_1	12+91.3	3.3L	1395.3	30E	0-50	16.0	15	1.6	2/10/84	43.1	15	6/1:1	
12+91.9	Q_2	12+92.1	2.2L	1395.3	30E	0-50	7.3	15	0.7	2/13/84	3.3	15	6:1	
12+92.5	T	12+92.7	1.0L	1395.1	30E	0-50	17.0	15	1.7	2/9/84	19.1	15	6/3:1	
12+93.8	Q_1	12+94	2.5L	1394.8	30E	0-50	6.4	15	0.6	2/10/84	1.5	15	6:1	
12+95	S	12+95.6	1.4L	1394.8	30E	0-50	17.0	15	1.7	2/3/84	30.5	15	6/2:1	
12+97.5	T	12+98.5	1.5L	1394.5	30E	0-50	9.2	15	0.9	2/9/84	1.0	15	6:1	
13+00	P	13+00.5	1.0L	13[illegible].3	30E	0-25	0.8	10	0.2	1/24/84	0.4	10	6:1	
						25-50	16.3	15	1.6	1/30/84	59.3	15	6/1:1	
13+02.5	T	13+03	1.0L	1394.2	30E	0-50	15.0	14	1.6	2/24/84	11.2	14	6/3:1	
13+05	S	13+05.2	1.0L	1393.6	30E	0-50	7.9	15	0.8	2/3/84	1.0	15	6:1	
13+10	P	13+08.5	0.7L	1393.9	30E	0-25	0.1	10	0.02	d				
						25-50	1.9	10	0.2	1/25/84	1.0	15	6:1	
13+20	P	13+18.5	1.8L	1394.1	30E	0-25	2.2	10	0.5	1/20/84	0.8	10	6:1	
						25-50	0.8	15	0.08	1/25/84	0.3	15	6:1	
13+30	P	13+28.5	1.6L	1393.3	30E	0-25	4.4	10	0.9	1/20/84	2.5	10	6:1	
						25-50	12.6	15	1.3	1/25/84	1.9	15	6:1	
103C	E	13+29	2.7L	1393.0	vert.	0-37.8	19.5	20	-	2/21/84	73.2	15	6/2:1	lost circulation during drilling at 36.3 feet
						37.8-45.0	0.0	15	0.0	d				
13+40	P	13+37.5	1.2L	1391.0	30E	0-25	10.1	10	2.1	1/20/84	4.4	10	6:1	
						25-50	7.5	15	0.7	1/25/84	1.4	15	6:1	
13+41.3	Q_1	13+38.6	2.1L	1391.2	30E	0-50	2.5	15	0.2	2/10/84	0.8	15	6:1	
13+42.5	T	13+39.7	1.0L	1391.4	30E	0-50	17.0	15	1.7	2/8/84	35.1	15	6/2:1	
13+43.8	Q_1	13+41.5	2.2L	1391.0	30E	0-50	11.4	15	1.1	2/10/84	3.5	15	6:1	
13+45	S	13+42	1.4L	1390.8	30E	0-25	0.5	10	0.1	1/31/84	1.0	10	6:1	
						25-50	11.2	15	1.1	2/3/84	38.3	15	6/2:1	

SEE SHEETS 14 & 15 FOR EXPLANATORY NOTES.

TABLE 8. NEW RIVER DAM, ARIZONA – FOUNDATION GROUTING SUMMARY

HOLE NO.	SERIES	HOLE LOCATION DAM STATION	HOLE LOCATION OFFSET	G.S. ELEV. (ft.)	INCLINATION/BEARING	DEPTH (ft.)	PRESSURE TESTING FLOW (gpm)	PRESSURE TESTING PRESS. (psi)	PRESSURE TESTING K VALUE (ft./day)	DATE GROUTED	GROUTING TAKE (sacks)	GROUTING PRESS (psi)	GROUTING MIX (W/C)	REMARKS
13+45.6	Q_2	13+42.7	1.3L	1390.7	30E	0-50	17.0	14	1.8	2/24/84	4.2	14	6/5:1	
13+46.3	Q_1	13+43.2	1.5L	1390.5	30E	0-50	15.5	15	1.5	2/10/84	8.3	15	6/4:1	
13+46.9	Q_2	13+44.2	2.7L	1390.4	30E	0-50	7.0	14	0.7	2/24/84	9.0	14	6/4:1	
13+47.5	T	13+45.5	1.7L	1390.0	30E	0-50	17.5	15	1.7	2/8/84	41.4	15	6/1½:1	
13+48.8	Q_1	13+45.9	0.7L	1389.9	30E	0-50	3.0	15	0.3	2/10/84	0.9	15	6:1	
13+50	P	13+47	1.2L	1389.4	30E	0-25	13.8	10	2.9	1/19-20 '84	31.3	10	6/2:1	placed 30.5 sacks on 1/19 before darkness halted work; washed hole and placed 0.8 sacks @ 6:1 on 1/20/84
						25-50	17.0	15	1.7	1/25/84	33.2	15	6/3:1	
13+52.5	T	13+47.5	2.6L	1389.3	30E	0-50	3.0	14	0.3	2/24/84	1.1	14	6:1	
13+55	S	13+52	0.8L	1388.2	30E	0-25	8.6	10	1.8	1/31/84	2.9	10	6:1	
						25-50	6.2	15	0.6	2/3/84	2.3	15	6:1	
13+60	P	13+57.5	1.6L	1387.5	30E	0-25	0.5	10	0.1	1/19/84	0.8	10	6:1	
						25-50	1.6	15	0.2	1/30/84	0.7	15	6:1	
13+70	P	13+67.4	1.5L	1386.5	30E	0-25	0.0	10	0.0	d				
						25-50	0.3	15	0.03	d				
13+80	P	13+77.3	0.8L	1384.5	30E	0-25	0.0	10	0.0	1/19/84	0.0	10	6:1	
						25-50	3.1	15	0.3	1/30/84	0.3	15	6:1	
13+85	S	13+84.5	1.3L	1386.0	30E	0-50	2.9	15	0.3	2/2/84	1.0	15	6:1	
13+90a	P	13+90	1.1L	1383.5	30E	0-25	7.0	10	1.4	1/19/84	0.0	10	6:1	
						25-50	16.0	15	1.6	1/27/84	43.2	15	6/2:1	
13+90b	P	13+90.4	2.2L	1383.5	vert.	0-50	4.3	15	0.4	2/14/84	0.5	15	6:1	
13+95	S	13+94.3	0.7L	1384.0	30E	0-50	0.0	15	0.0	d				
14+00a	P	14+00	1.1L	1384.7	30E	0-25	2.2	10	0.5	1/19/84	0.7	10	6:1	
						25-50	15.0	15	1.5	1/27/84	7.9	15	6:1	
14+00b	P	14+00	2.6L	1384.7	vert.	0-50	0.7	15	0.06	d				
14+00c	P	14+00	0.3R	1384.7	30W	0-50	0.0	15	0.0	d				
14+05	S	14+04.3	0.5L	1385.2	30E	0-50	1.4	15	0.1	2/2/84	0.7	15	6:1	

SEE SHEETS 14 & 15 FOR EXPLANATORY NOTES.

TABLE 8. NEW RIVER DAM, ARIZONA – FOUNDATION GROUTING SUMMARY

HOLE NO.	SE-RIES	HOLE LOCATION		G.S. ELEV. (ft)	INCLI-NATION/ BEARING	DEPTH (ft.)	PRESSURE TESTING			DATE GROUTED	GROUTING			REMARKS
		DAM STATION	OFF-SET				FLOW (gpm)	PRESS (psi)	K VALUE (ft./day)		TAKE (sacks)	PRESS (psi)	MIX (W/C)	
14+10a	P	14+09.5	3.7L	1385.5	30E	0-25	0.0	10	0.0	d				
						25-50	0.0	15	0.0	d				
14+10b	P	14+11	1.0L	1385.6	vert.	0-25	0.0	10	0.0	d				
						25-50	0.9	15	0.08	1/27/84	0.6	15	6:1	
14+10c	P	14+11.4	2.2R	1385.6	30W	0-25	0.0	10	0.0	d				
						25-50	5.1	15	0.5	1/27/84	2.4	15	6:1	
14+20	P	14+21.5	1.0L	1385.9	vert.	0-15	0.0	5	0.0	d				
14+30	P	14+30	1.0L	1384.6	vert.	0-30	0.0	10	0.0	d				
14+40	P	14+40	1.0L	1384.1	vert.	0-15	0.1	5	0.06	d				
14+50	P	14+50	1.0R	1388.4	30E	0-25	0.0	10	0.0	d				
						25-50	0.1	15	0.01	d				
14+60	P	14+60	1.0R	1388.5	30E	0-25	0.0	10	0.0	d				
						25-50	0.1	15	0.01	d				
14+70	P	14+70	1.0R	1388.6	30E	0-25	0.0	10	0.0	d				
						25-50	0.1	15	0.01	d				
14+80	P	14+80	1.0R	1388.3	30E	0-25	0.0	10	0.0	d				
						25-50	0.1	15	0.01	d				
14+90	P	14+90	1.0R	1388.0	30E	0-25	0.0	10	0.0	d				
						25-50	0.3	15	0.03	d				
15+00	P	15+00	1.0R	1387.4	30E	0-25	0.7	10	0.1	2/9/84	0.2	10	6:1	
						25-50	4.5	15	0.4	2/13/84	1.1	15	6:1	
15+10	P	15+10	1.0R	1387.1	30E	0-25	0.0	10	0.0	d				
						25-50	3.8	15	0.4	2/13/84	1.3	15	6:1	
15+20	P	15+20	1.0R	1386.9	30E	0-25	0.0	10	0.0	d				
						25-50	4.2	15	0.4	2/13/84	3.6	15	6:1	
15+30	P	15+30	1.0R	1386.4	30E	0-25	0.0	10	0.0	d				
						25-50	1.5	15	0.1	2/13/84	0.3	15	6:1	
15+40	P	15+40	1.0R	1387.6	30E	0-25	0.0	10	0.0	d				
						25-50	0.0	15	0.0	d				
15+50	P	15+49	1.0R	1389.1	30E	0-25	0.0	10	0.0	d				
						25-50	0.0	15	0.0	d				

SEE SHEETS 14 & 15 FOR EXPLANATORY NOTES.

TABLE 8. NEW RIVER DAM, ARIZONA – FOUNDATION GROUTING SUMMARY

HOLE NO.	SERIES	HOLE LOCATION		G.S. ELEV. (ft.)	INCLINATION/ BEARING	DEPTH (ft.)	PRESSURE TESTING			DATE GROUTED	GROUTING			REMARKS
		DAM STATION	OFF-SET				FLOW (gpm)	PRESS. (psi)	K VALUE (ft./day)		TAKE (sacks)	PRESS (psi)	MIX (W/C)	
15+60	P	15+60	1.0R	1388.7	30E	0-25	3.2	10	0.7	2/10/84	0.9	10	6:1	zone 3 drilled for exploratory purposes
						25-50	6.2	15	0.6	2/16/84	1.8	15	6:1	
						50-75	0.7	29	0.04	d				
15+70	P	15+70	1.0R	1388.6	30E	0-25	5.7	10	1.2	2/10/84	1.5	10	6:1	
						25-50	3.7	15	0.4	2/15/84	1.7	15	6:1	
15+80	P	15+80	1.0R	1388.2	30E	0-25	6.8	10	1.4	2/10/84	4.9	10	6:1	
						25-50	5.5	15	0.5	2/15/84	5.8	15	6:1	
15+90	P	15+90	1.0R	1388.1	30E	0-25	6.8	10	1.4	2/10/84	3.1	10	6:1	
						25-50	5.3	15	0.5	2/15/84	1.1	15	6:1	
16+00	P	16+00	2.0R	1388.9	30E	0-25	0.0	10	0.0	d				
						25-50	2.0	15	0.2	2/15/84	1.1	15	6:1	
16+05	S	16+05	1.0R	1388.7	30E	0-25	5.5	10	1.1	2/15/84	1.1	10	6:1	
16+10	P	16+10	1.0R	1388.5	30E	0-25	9.4	10	1.9	2/10/84	9.9	10	6:1	
						25-50	9.3	10	1.1	3/14/84	4.2	6	6/5:1	
16+15	S	16+15	1.0R	1388.5	30E	0-25	8.8	10	1.8	2/15/84	2.9	10	6:1	
16+20	P	16+20	2.0R	1388.4	30E	0-25	12.5	10	2.6	2/9/84	17.3	10	6/4:1	
						25-50	0.1	7	0.01	d				
16+21.3	Q_1	16+20.8	2.0R	1388.4	30E	0-25	4.0	7	1.0	3/13/84	0.7	6	6:1	
16+22.5	T^1	16+22	2.0R	1388.2	30E	0-25	1.8	10	0.4	2/16/84	33.2	10	6/2½:1	
16+23.8	Q_1	16+23.3	2.0R	1388.2	30E	0-25	2.3	7	0.6	3/13/84	1.3	6	6:1	
16+25	S^1	16+24.5	2.0R	1388.0	30E	0-25	13.8	10	2.9	2/15/84	30.1	10	6/2:1	
16+27.5	T	16+28	2.0R	1387.8	30E	0-25	5.4	10	1.1	2/16/84	6.6	10	6:1	
16+30	P	16+30	2.0R	1387.6	30E	0-25	12.2	10	2.5	2/ 9/84	14.2	10	6/4½:1	
						25-50	0.8	7	0.1					
16+35	S	16+35	2.0R	1387.4	30E	0-25	1.9	10	0.4	2/15/84	0.8	10	6:1	
						25-50	2.3	15	0.2	3/15/84	1.0	14	6:1	
16+40	P	16+40	2.5R	1386.0	30E	0-25	1.3	11	0.3	3/13/84	0.7	9	6:1	
						25-50	0.6	7	0.08	d				
16+50	P	16+49.5	2.5R	1383.1	30E	0-25	0.8	7	0.2	3/13/84	0.8	6	6:1	
						25-50	1.6	15	0.2	3/15/84	0.5	14	6:1	
16+60	P	16+59.5	2.8R	1378.7	30E	0-25	0.0	6	0.0	d				
						25-50	1.3	15	0.1	3/15/84	0.5	14	6:1	

SEE SHEETS 14 & 15 FOR EXPLANATORY NOTES.

TABLE 8. NEW RIVER DAM, ARIZONA – FOUNDATION GROUTING SUMMARY

HOLE NO.	SE-RIES	HOLE LOCATION		G.S ELEV (ft)	INCLI-NATION/ BEARING	DEPTH (ft.)	PRESSURE TESTING			DATE GROUTED	GROUTING			REMARKS
		DAM STATION	OFF-SET				FLOW (gpm)	PRESS (psi)	K VALUE (ft./day)		TAKE (sacks)	PRESS (psi)	MIX (W/C)	
16+70	P	16+68.5	2.3R	1374.2	30E	0-25	0.1	8	0.02	d				
						25-50	0.6	15	0.06	d				
16+80	P	16+78.5	1.0R	1371.9	30E	0-25	0.8	7	0.2	3/13/84	0.5	6	6:1	
						25-50	1.3	15	0.1	3/15/84	1.0	14	6:1	
16+90	P	16+88	0	1372.0	30E	0-25	0.0	6	0.0	d				
						25-50	0.8	15	0.08	d				
17+00	P	16+99	0	1372.5	30E	0-25	0.0	7	0.0	d				
						25-50	0.0	15	0.0	d				
17+10	P	17+09	0	1371.9	30E	0-25	0.1	7	0.03	d				
						25-50	0.7	15	0.07	d				
17+20	P	17+20	0	1371.4	30E	0-25	0.7	7	0.2	3/14/84	0.3	6	6:1	
						25-50	0.9	15	0.09	d				
17+30	P	17+30	0	1370.7	30E	0-25	0.0	7	0.0	d				
						25-50	0.0	15	0.0	d				
17+40	P	17+40.5	0	1372.0	30E	0-25	0.0	7	0.0	d				
						25-50	0.0	15	0.0	d				
17+50	P	17+49	0	1372.3	30E	0-25	0.0	7	0.0	d				
						25-50	0.0	15	0.0	d				
104C	E	17+49	20L	1373.3	vert.	0-46.1	0.0	7	0.0	d				
17+60	P	17+59	0	1373.8	30E	0-25	0.0	7	0.0	d				
						25-50	0.2	15	0.02	d				
17+70	P	17+69	0	1374.6	30E	0-25	0.9	7	0.2	3/13/84	0.3	6	6:1	
						25-50	0.9	15	0.09	d				
17+80	P	17+79	1.0R	1375.0	30E	0-25	1.9	7	0.5	3/13/84	0.3	6	6:1	
						25-50	0.7	15	0.07	d				
17+90	P	17+90	2.0R	1375.6	30E	0-25	0.0	7	0.0	d				
						25-50	1.7	15	0.2	3/16/84	0.3	14	6:1	
18+00	P	17+99	1.0R	1375.7	30E	0-25	2.8	7	0.7	3/13/84	0.4	6	6:1	
						25-50	0.8	15	0.08	d				
18+10	P	18+09	0	1375.1	30E	0-25	[illegible].1	7	2.3	3/15/84	2.7	6	6:1	
						25-50	1.7	15	0.2	3/16/84	0.4	14	6:1	
18+20	P	18+20	0	1374.7	30E	0-25	0.9	7	0.2	3/15/84	0.3	6	6:1	
						25-50	0.5	15	0.05	d				

SEE SHEETS 14 & 15 FOR EXPLANATORY NOTES.

TABLE 8. NEW RIVER DAM, ARIZONA - FOUNDATION GROUTING SUMMARY

HOLE NO.	SERIES	HOLE LOCATION: DAM STATION	HOLE LOCATION: OFFSET	G.S. ELEV. (ft)	INCLINATION/ BEARING	DEPTH (ft.)	PRESSURE TESTING: FLOW (gpm)	PRESSURE TESTING: PRESS. (psi)	PRESSURE TESTING: K VALUE (ft./day)	DATE GROUTED	GROUTING: TAKE (sacks)	GROUTING: PRESS (psi)	GROUTING: MIX (W/C)	REMARKS
18+30	P	18+30	0	1374.0	30E	0-25	0.0	7	0.0	d				
						25-50	1.0	15	0.1	d				
18+40	P	18+40	0	1373.5	30E	0-25	0.0	7	0.0	d				
						25-50	2.6	15	0.3	3/16/84	0.5	14	6:1	
18+50	P	18+51	0	1372.8	30E	0-25	0.0	7	0.0	d				
						25-50	1.9	17	0.2	3/16/84	0.6	14	6:1	
18+60	P	18+61	0	1371.3	30E	0-25	1.4	7	0.4	3/15/84	1.0	6	6:1	
						25-50	2.9	15	0.3	3/16/84	0.5	14	6:1	
18+70	P	18+70	0	1368.4	30E	0-25	0.0	7	0.0	d				
						25-50	7.7	20	0.7	3/16/84	0.6	14	6:1	
18+80	P	18+80.5	0	1367.4	30E	0-25	0.0	7	0.0	d				
						25-50	0.8	15	0.08	d				
18+90	P	18+90	0	1366.5	30E	0-25	0.0	7	0.0	d				
						25-50	0.5	15	0.05	d				
19+00	P	19+00	0	1368.2	30E	0-25	0.0	7	0.0	d				
						25-50	0.2	15	0.02	d				
19+10	P	19+10	0.5R	1369.0	30E	0-25	0.0	7	0.0	d				zone 3 drilled
						25-50	0.1	15	0.01	d				for exploratory
						50-75	0.6	29	0.03	d				purposes
19+20	P	19+20	1.0R	1369.4	30E	0-25	0.0	7	0.0	d				
						25-50	0.0	15	0.0	d				
19+30	P	19+30	1.0R	1369.8	30E	0-25	0.0	7	0.0	d				
						25-50	0.0	15	0.0	d				
19+40	P	19+40	0.5R	1369.2	30E	0-25	0.0	7	0.0	d				
						25-50	0.5	15	0.05	d				
19+50	P	9+49	0	1369.7	30E	0-25	0.0	7	0.0	d				
						25-50	0.0	15	0.0	d				
19+60	P	19+60	0	1368.8	30E	0-25	0.0	7	0.0	d				
						25-50	0.4	15	0.04	d				
19+70	P	19+70	0	1369.2	30E	0-25	0.0	7	0.0	d				
						25-50	0.7	15	0.07	d				
19+80	P	19+80	0	1368.7	30E	0-25	0.0	7	0.0	d				
						25-50	0.9	15	0.09	d				

SEE SHEETS 14 & 15 FOR EXPLANATORY NOTES.

TABLE 8. NEW RIVER DAM, ARIZONA - FOUNDATION GROUTING SUMMARY

HOLE NO	SERIES	HOLE LOCATION		G.S. ELEV (ft.)	INCLINATION/ BEARING	DEPTH (ft.)	PRESSURE TESTING			DATE GROUTED	GROUTING			REMARKS
		DAM STATION	OFF-SET				FLOW (gpm)	PRESS (psi)	K VALUE (ft./day)		TAKE (sacks)	PRESS (psi)	MIX (W/C)	
19+90	P	19+90	0	1368.3	30E	0-25	0.0	7	0.0	d				
						25-50	0.8	15	0.08	d				
20+00	P	20+00	0	1367.9	30E	0-25	0.0	7	0.0	d				
						25-50	0.1	15	0.01	d				
20+20	P	20+20	0	1368.3	30E	0-25	0.0	7	0.0	d				
						25-50	0.5	15	0.05	d				
20+40	P	20+40	0	1368.2	30E	0-25	0.1	7	0.03	d				
						25-50	0.0	15	0.0	d				
20+60	P	20+60	0	1367.4	30E	0-25	0.2	7	0.05	d				
						25-50	0.4	14	0.04	d				
20+80	P	20+80	0	1366.6	30E	0-25	0.4	7	0.1	d				
						25-50	0.0	15	0.0	d				
20+99	P	20+99	1.0L	1366.5	30W	0-25	0.0	7	0.0	d				
						25-50	0.1	15	0.01	d				
21+00	P	21+00	1.0R	1366.5	30E	0-25	0.0	7	0.0	d				
						25-50	0.0	15	0.0	d				
21+01	P	21+01	0	1366.5	vert.	0-25	0.2	7	0.05	d				
						25-50	0.2	15	0.02	d				
														alluvial foundation between stations 21+06 and 29+64
29+70	P	29+69	7.0L	1356.0	15E	0-25	0.0	15	0.0	d				
						25-50	11.0	15	1.0	1/13/84	27.0	15	6/2:1	
29+72.5	T	29+71.5	5.0L	1355.5	12½E	0-50	0.0	10	0.0	d				
29+75	S	29+74	6.0L	1355.5	10E	0-50	10.9	10	1.2	1/18/84	49.4	15	6/2:1	
29+77.5	T	29+77	5.0L	1354.5	5E	0-50	0.0	10	0.0	d				
						50-75	16.9	25	0.9	1/27/84	51.6	25	6/1½:1	
29+78.3	Q_1	29+78.5	6.0L	1354.5	2½E	0-75	0.5	25	0.03	d				
101C	E	29.79	5.5L	1353.5	vert.	0-66	4.9	10	0.3	2/8/84	2.1	10	6:1	tight 23-66ft.
29+80	P	29+80	7.0L	1352.5	vert.	0-25	0.0	15	0.0	d				
						25-50	0.5	15	0.05	1/13/84	1.3	15	6:1	
						50-75	16.3	25	0.9	1/24/84	62.3	25	6/1:1	

SEE SHEETS 14 & 15 FOR EXPLANATORY NOTES.

TABLE 8. NEW RIVER DAM, ARIZONA – FOUNDATION GROUTING SUMMARY

HOLE NO.	SERIES	HOLE LOCATION		G S ELEV (ft)	INCLINATION/ BEARING	DEPTH (ft)	PRESSURE TESTING			DATE GROUTED	GROUTING			REMARKS
		DAM STATION	OFF-SET				FLOW (gpm)	PRESS (psi)	K VALUE (ft./day)		TAKE (sacks)	PRESS (psi)	MIX (W/C)	
29+82.5	T	29+81	8.0L	1353.3	7½W	0-50	0.2	15	0.02	d				
						50-75	0.1	25	0.01	d				
29+85	S	29+85	6.0L	1352.5	15W	0-50	5.4	15	0.5	1/18/84	0.6	15	6:1	
						50-75	5.1	25	0.3	1/27/84	5.2	25	6:1	
29+87.5	T	29+88	5.5L	1352.5	22½W	0-50	0.1	15	0.01	d				
						50-75	0.3	25	0.02	d				
29+90	P	29+92	4.8L	1351.6	30W	0-25	0.0	10	0.0	d				
						25-50	2.7	15	0.3	1/14/84	10.1	15	6/4:1	
						50-75	7.8	25	0.5	1/24/84	10.7	25	6:1	
29+95	S	29+97	4.5L	1352.1	30W	0-50	2.0	15	0.2	1/18/84	0.9	15	6:1	
30+00	P	30+00	3.8R	1351.2	30W	0-25	0.0	10	0.0	d				
						25-50	4.0	15	0.4	1/14/84	2.2	15	6:1	
						50-75	7.1	25	0.4	1/24/84	2.9	25	6:1	
30+10	P	30+11.5	4.5R	1351.3	30W	0-25	0.0	15	0.0	d				
						25-50	1.5	15	0.1	1/14/84	0.3	15	6:1	
						50-75	13.0	25	0.8	1/27/84	15.3	25	6/4:1	
30+15	S	30+15	5.0R	1352.1	30W	0-75	0.4	25	0.02	d				
30+20	P	30+20	4.1R	1352.8	30W	0-25	0.0	14	0.0	d				
						25-50	0.0	15	0.0	d				
						50-75	19.2	25	1.1	1/24/84	86.1	25	6/1:1	
30+30	P	30+31.5	2.6L	1352.8	30W	0-25	0.0	15	0.0	d				
						25-50	1.4	15	0.1	1/14/84	0.3	15	6:1	
						50-75	3.1	25	0.2	1/28/84	1.6	25	6:1	
30+40	P	30+40	3.6L	1353.0	30W	0-25	0.0	15	0.0	d				
						25-50	1.1	15	0.1	1/14/84	0.3	15	6:1	
						50-75	20.1	25	1.2	1/25/84	72.8	25	6/1:1	
30+50	P	30+49	4.4L	1351.7	30W	0-25	0.0	8	0.0	d				
						25-50	4.0	15	0.4	1/14/84	1.9	15	6:1	
						50-75	0.0	25	0.0	d				
30+60	P	30+58.5	3.6R	1352.0	30W	0-25	0.0	10	0.0	d				
						25-50	5.4	15	0.5	1/14/84	2.0	15	6:1	
						50-75	18.3	25	1.1	1/25/84	80.1	25	6/1:1	

SEE SHEETS 14 & 15 FOR EXPLANATORY NOTES.

TABLE 8. NEW RIVER DAM, ARIZONA - FOUNDATION GROUTING SUMMARY

HOLE NO	SERIES	HOLE LOCATION		G S ELEV (ft)	INCLINATION/ BEARING	DEPTH (ft)	PRESSURE TESTING			DATE GROUTED	GROUTING			REMARKS
		DAM STATION	OFFSET				FLOW (gpm)	PRESS (psi)	K VALUE (ft / day)		TAKE (sacks)	PRESS (psi)	MIX (W/C)	
30+65	S	30+68	0	1351.9	30W	0-75	0.7	25	0.04	d				
30+70	P	30+72	0	1351.7	30W	0-25	0.0	16	0.0	d				
						25-50	0.3	15	0.03	d				
						50-75	7.5	25	0.4	1/28/84	9.5	25	6/5:1	
30+75	S	30+76	0	1352.8	30W	0-75	0.5	25	0.03	d				
30+80	P	30+78.5	3.0L	1353.0	30W	0-25	0.0	19	0.0	d			-	
						25-50	2.1	15	0.2	1/16/84	0.1	15	6:1	
						50-75	14.4	25	0.9	1/25/84	54.3	25	6/1:1	
30+90	P	30+91	0	1354.1	30W	0-25	0.0	10	0.0	d				
						25-50	2.3	15	0.2	1/16/84	2.1	15	6:1	
						50-75	1.5	25	0.09	d				
31+00	P	31+00.5	0	1354.5	30W	0-25	0.0	10	0.0	d				see sheet 13 for
						25-50	2.1	15	0.2	1/16/84	2.7	15	6:1	grouting record
						50-75	7.5	25	0.4	1/25/84	15.8	25	6/3:1	of hole 31+10P
31+15	S	31+15	2.3L	1354.4	30W	0-50	1.2	15	0.1	1/18/84	0.7	15	6:1	
31+20	P	31+20	2.6L	1355.5	30W	0-25	0.0	12	0.0	d				
						25-50	11.0	15	1.1	1/16/84	16.0	15	6/4:1	
						50-75	2.2	25	0.1	1/25/84	2.4	25	6:1	
31+25	S	31+25.5	3.1L	1355.8	30W	0-50	0.0	10	0.0	d				
100C	E	31+28.5	6.0L	1354.9	vert	0-66.6	6.5	15	0.3	2/6/84	7.4	10	6/5:1	
31+30	P	31+31	4.9L	1355.4	30W	0-25	0.0	13	0.0	d				
						25-50	6.7	15	0.7	1/20/84	17.8	15	6/3:1	
						50-75	0.0	25	0.0	d				
31+35	S	31+36	4.4L	1354.8	30W	0-50	0.3	10	0.04	d				
31+40	P	31+43.5	0	1353.1	30W	0-25	0.7	10	0.1	1/12/84	0.1	10	6:1	
						25-50	2.0	15	0.2	1/20/84	0.6	15	6:1	
						50-75	13.2	25	0.8	1/26/84	21.6	25	6/4:1	
31+50	P	31+50.5	0	1355.7	30W	0-25	5.0	10	1.0	1/12/84	2.1	10	6:1	
						25-50	1.8	15	0.2	1/21/84	0.7	15	6:1	
						50-75	0.2	25	0.01	d				
31+60	P	31+60	0.7R	1359.7	30W	0-25	0.0	10	0.0	1/12/84	0.2	10	6:1	grouted by acci-
						25-50	7.2	15	0.7	1/21/84	5.5	15	6:1	dent
						50-75	16.4	25	1.0	1/26/84	57.2	25	6/1½:1	

SEE SHEETS 14 & 15 FOR EXPLANATORY NOTES.

TABLE 8. NEW RIVER DAM, ARIZONA – FOUNDATION GROUTING SUMMARY

HOLE NO.	SERIES	HOLE LOCATION		G.S. ELEV (ft.)	INCLINATION/ BEARING	DEPTH (ft.)	PRESSURE TESTING			DATE GROUTED	GROUTING			REMARKS
		DAM STATION	OFFSET				FLOW (gpm)	PRESS (psi)	K VALUE (ft./day)		TAKE (sacks)	PRESS (psi)	MIX (W/C)	
31+70	P	31+70	0	1365.0	30W	0-25	0.0	12	0.0	1/12/84	0.1	5	6:1	grouted zone 1
						25-50	5.8	15	0.6	1/21/84	3.6	15	6:1	due to initial
						50-75	0.1	25	0.01	d				high water takes
31+80	P	31+79	0	1372.2	30W	0-25	4.5	5	1.4	1/17/84	0.2	5	6:1	
						25-50	2.6	15	0.3	1/21/84	1.5	15	6:1	
						50-75	3.0	25	0.2	1/26/84	1.1	25	6:1	
DD-23		31.86	2.2R	1379.2	vert	0-25	0.0	10	0.0	d				
31+90	P	31+87	0	1380.0	30W	0-25	0.2	5	0.06	d				
						25-50	4.2	15	0.4	1/21/84	1.2	15	6:1	
						50-75	4.5	25	0.3	1/28/84	0.8	25	6:1	
31+95	P	31+94.5	2.0L	1386.4	30W	0-25	8.6	5	2.7	1/17/84	3.3	5	6/5:1	
						25-50	3.1	15	0.3	1/21/84	0.6	15	6:1	
						50-75	21.6	25	1.3	1/26/84	50.6	25	6/1:1	
DD-22		31+96	3.8R	1387.8	60W	0-25	0.1	10	0.02	d				
32+00	P	32+01	1.5R	1392.3	30W	0-25	0.0	5	0.0	d				
						25-50	4.1	15	0.4	1/21/84	1.1	15	6:1	
						50-75	4 9	25	0.3	1/28/84	5.9	25	6/5:1	
32+05	S	32+05.5	0	1397.5	30W	0-25	7.2	10	1.5	1/26/84	6.5	10	6/5:1	
						25-50	0.0	15	0.0	d				
32+10	P	32+09.5	0	1400.5	30W	0-25	9.2	5	2.8	1/17/84	40.7	5	6/2:1	
						25-50	14.7	15	1.5	1/21/84	77.2	15	6/1:1	
						50-75	0.1	25	0.01	d				
32+15	S	32+13.5	0	1403.7	30W	0-25	6.1	10	1.3	1/26/84	3.2	10	6:1	packer leaked
32+17	P	32+17.5	0	1406.4	30W	0-25	8.5	10	1.8	1/23/84	13.0	10	6/5:1	grout leak at
						25-50	3.3	15	0.3	3/7/84	2.4	15	6:1	sta. 32+21.5, 21, after 1 hr; leak sealed; no further
32+21	S	32+21	0	1408.7	30W	0-25	0.5	10	0.1	3/6/84	0.0	7	6:1	grout take
32+24	P	32+24	2.0R	1408.9	30W	0-25	surface leaks			3/2/84	2.2	8	6:1	surface leaks
						25-50	0.0	15	0.0	d				
32+27	S	32+27	0	1414.6	30W	0-25	0.1	10	0.02	d				
32+30	P	32+30	0	1416.5	30W	0-25	surface leaks			3/2/84	12.6	8	6/3:1	surface leaks
						25-50	3.1	15	0.3	3/7/84	1.4	15	6:1	

SEE SHEETS 14 & 15 FOR EXPLANATORY NOTES.

TABLE 8. NEW RIVER DAM, ARIZONA – FOUNDATION GROUTING SUMMARY

HOLE NO.	SERIES	HOLE LOCATION		G.S. ELEV. (ft.)	INCLINATION/ BEARING	DEPTH (ft.)	PRESSURE TESTING			DATE GROUTED	GROUTING			REMARKS
		DAM STATION	OFFSET				FLOW (gpm)	PRESS. (psi)	K VALUE (ft./day)		TAKE (sacks)	PRESS. (psi)	MIX (W/C)	
32+34	S	32+34	0	1419.1	30W	0-25	0.8	10	0.2	3/6/84	3.3	7	6/5:1	
32+37	P	32+37.5	1.5R	1419.8	30W	0-25	surface leaks			3/2/84	4.4	6	6/5:1	
						25-50	0.0	20	0.0	d				
32+45	P	32+45	0	1428.5	30W	0-25	6.5	10	1.3	3/2/84	0.3	6	6:1	
						25-50	5.8	16	0.6	3/7/84	4.0	15	6/5:1	
32+52	P	32+52	0	1433.5	30W	0-25	1.2	7	0.3	3/5/84	0.2	7	6:1	grout leak
						25-50	6.8	18	0.7	3/7/84	2.8	15	6:1	
32+60	P	32+60	0	1438.4	30W	0-25	6.0	10	1.2	3/5/84	0.2	7	6:1	leak at sta. 32+62
						25-50	1.0	15	0.09	d				
32+67	P	32+67	0	1441.7	30W	0-25	surface leaks			3/5/84	0.6	7	6:1	
32+75	P	32+75	0	1445.0	30W	0-25	6.4	10	1.3	3/5/84	0.1	7	6:1	
32+79	S	32+79.5	0	1451.0	30W	0-25	0.1	10	0.02	d				
32+82	P	32+82	0	1452.5	30W	0-25	surface leaks			3/5/84	0.1	7	6:1	
32+86	S	32+86	0	1454.1	30W	0-25	0.2	10	0.04	d				
32+90	P	32+90	0	1457.6	30W	0-25	2.1	8	0.5	3/5/84	0.1	7	6:1	
32+97	P	32+97	0	1460.2	30W	0-25	10.2	10	2.1	3/5/84	1.2	7	6:1	
33+05	P	33+05	0	1470.0	30W	0-25	12.0	10	2.5	3/5/84	1.8	7	6:1	tested with packer at 7 ft. due to surface leaks
33+11	P	33+11	0	1472.8	30W	0-25	0.8	7	0.2	3/5/84	0.2	7	6:1	tested with packer at 7 ft. due to surface leaks
33+17	P	33+17	0	1477.6	30W	0-25	0.0	7	0.0	d				
31+10	P	31+09.5	0	1354.5	30W	0-25	0.0	10	0.0	d				inadvertently omitted from sheet 11
						25-50	3.0	15	0.3	1/16/84	0.9	15	6:1	
						50-75	0.4	25	0.02	d				

SEE SHEETS 14 & 15 FOR EXPLANATORY NOTES.

TABLE 8. FOUNDATION GROUTING SUMMARY

(notes)

1. Hole numbers correspond to approximate dam stationing and were determined before final surveying.

2. DD-22 and DD-23 are pre-construction core holes which were incompletely backfilled after drilling.

3. Hole series designations are as follows: P = primary grout hole; S = secondary grout hole; T= tertiary grout hole; Q_1 = quaternary grout hole; Q_2 = quinary grout hole; E = exploratory grout (core) hole.

4. Grout hole locations are shown using dam stationing and feet offset (if any) left (upstream) and right (downstream) of dam centerline as determined by tape measure.

5. Grout hole elevations were determined from dam centerline survey data.

6. Grout hole inclinations are measured from the vertical.

7. Grout hole bearings, designated by an "E" or "W", generally parallel the dam axis, which is approximately N77^{o}E.

8. Only the lowermost zone in a split-spaced grout hole required pressure testing and possible grouting if the depth interval shown spans more than one zone.

9. Pressure test data is tabulated as follows: Flow (Q) is measured in gallons per minute. Pressure is the gage pressure in pounds per square inch measured at the top of the hole. Permeabilities (K values) for each grout hole depth interval are measured in feet per day as determined by the formula:

$$K = \frac{(30.64 \ln 9.6L)\ (Q)}{(L)\ (H_t)}$$

based on a hole diameter of 2-1/2 inches and where H_t equals the average vertical depth of the test interval (H_w) plus the water pressure at the top of the hole (H_p) in feet. H_p is determined by multiplying the gage pressure by 2.31. For NW size exploratory grout holes and core holes, the formula is:

$$K = \frac{(30.64 \ln 8L)\ (Q)}{(L)\ (H_t)}$$

TABLE 8. (Continued)

10. Permeabilities (K values) are not calculated where drill water return was lost because of the probable heterogeneous nature of the interval permeability.

11. In order to normalize permeabilities calculated for split-spaced grout holes, it was assumed that the K value shown reflects only the lowermost 25 ft. zone. In addition, is was assumed that any grout injection would occur only in that lowermost zone.

12. Depth intervals in which grouting was deferred are so designated by the letter "d".

13. Mix ratios are the volume proportion of dry cement: water. When more than one ratio is indicated, the grout was thickened incrementally within the range shown.

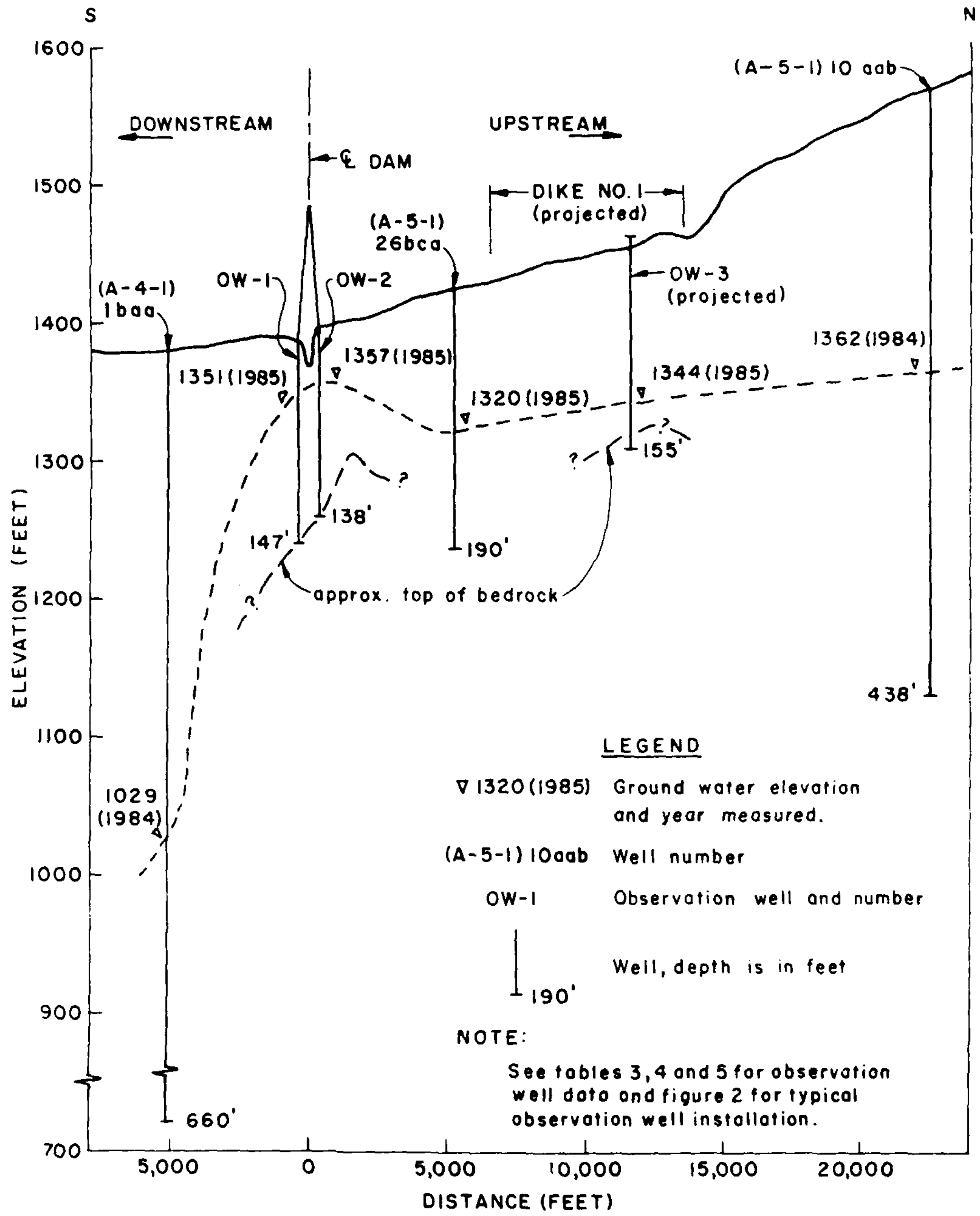


FIGURE 1. NEW RIVER DAM–GROUND WATER BASIN PROFILE

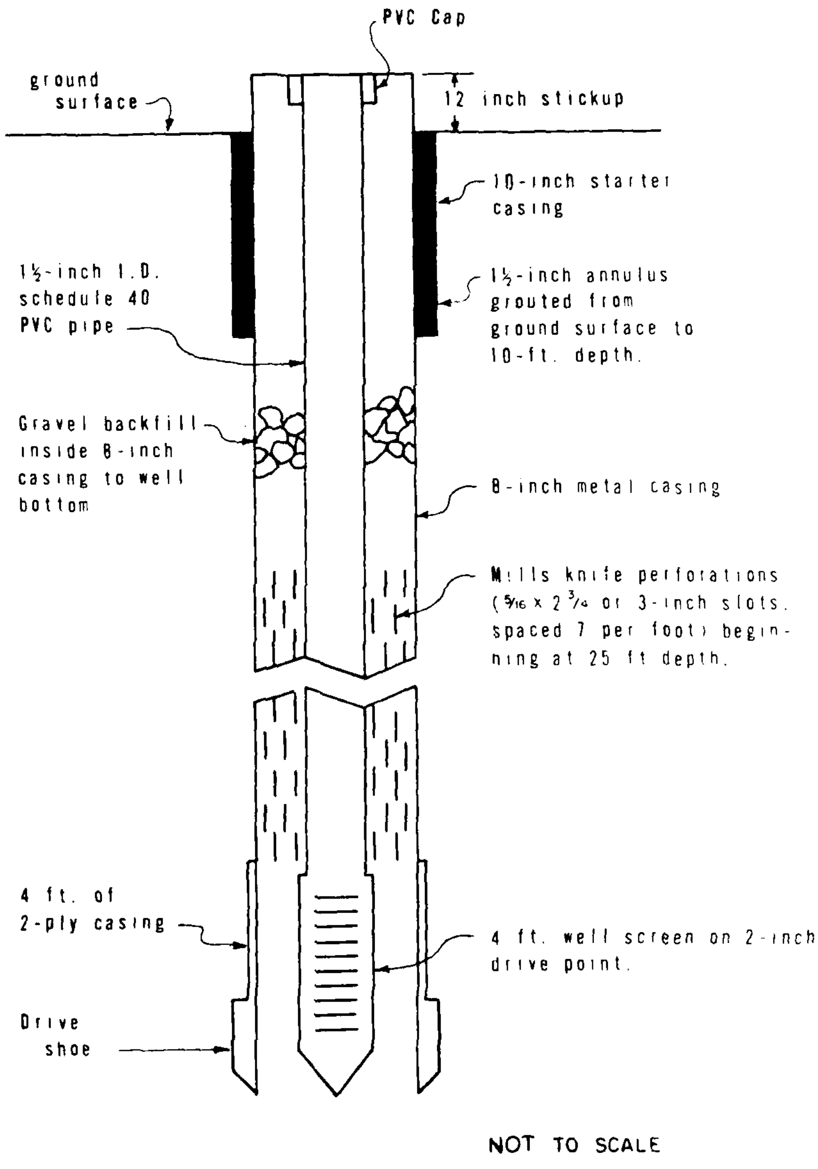
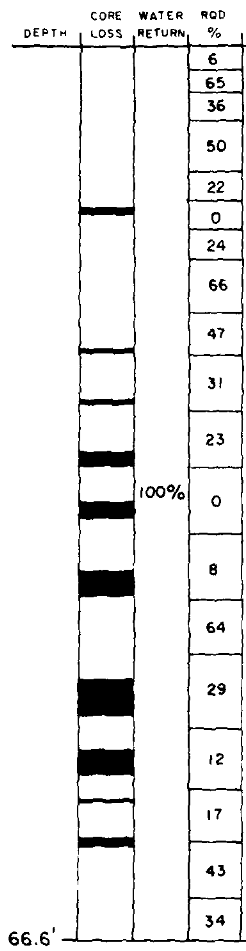


FIGURE 2. TYPICAL OBSERVATION WELL

100C

(STA. 31+28.5, 6'L)



ANDESITE (Tva): medium to dark gray, hard, generally unweathered, aphanitic, moderately to highly fractured, numerous shattered to brecciated zones from 2 to 12 inches thick between 10 and 60 feet; fracture surfaces are smooth to moderately rough, generally coated with black oxidation or reddish-brown to green clay; heavy clay coating between 0 and 3 feet, light clay coating between 3 and 15.5 feet and sporadic clay coating below 15.5 feet; calcite lined fractures present below 40 feet; fractures dip predominantly 45 to 80 degrees from the horizontal; 3-inch wide brecciated and rehealed fracture dipping 70 degrees between 42.1 and 44.0 feet; grout and clay present along maximum 1/4-inch wide fracture dipping 60 degrees between 61.6 and 62.2 feet.

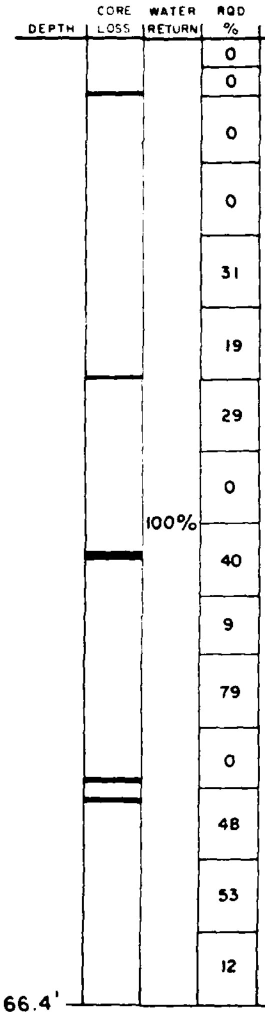
NOTES:

1. Total core recovery 85 percent.
2. See table 8, Foundation grouting summary, for pressure test and grouting data.

FIGURE 3. LOG OF EXPLORATORY CORE HOLE 100C.

101C

(STA. 29+79, 5.5' L)



EL 1353.5 DESCRIPTION

ANDESITE (Tva): medium to dark gray, hard, generally unweathered, aphanitic, highly fractured to shattered between 0 and 33.2 feet; moderately to highly fractured below 33.2 feet. approximately 25 percent of the core is shattered to brecciated in zones from 1 to 12 inches thick; fracture surfaces are smooth to moderately rough, generally coated with black oxidation or reddish-brown to green clay; heavy clay coating between 0 and 7 feet, light clay coating between 7 and 61 feet, clay absent below 61 feet; fractures dip predominantly 45 to 80 degrees from the horizontal; grout and clay partially to completely fill fractures of various orientations in highly fractured to shattered core between 48 and 52 feet; grout fills 1/8-inch wide, 60 degree dipping fracture between 55.2 and 55.4 feet, grout partially fills 1/8"-wide, 45 degree dipping fracture between 55.6 and 56.1 feet, grout seams and pockets maximum 1/4-inch wide in brecciated and rehealed zone between 56.5 and 61.1 feet.

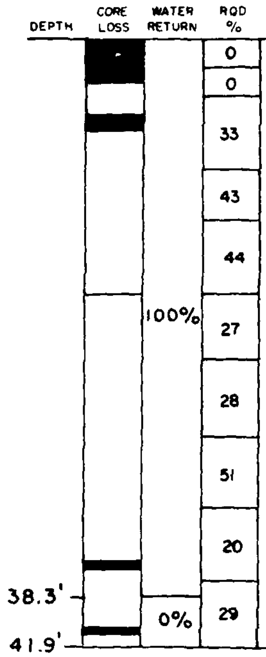
NOTES:

1. Total core recovery 99 percent.
2. See table 8, Foundation grouting summary, for pressure test and grouting data.

FIGURE 4. LOG OF EXPLORATORY CORE HOLE 101C.

102C

(STA. 12+78.5, 1.5 L)



EL 1397.3 DESCRIPTION

GRANITE (gr): light-gray to reddish-brown, medium to coarse grained, hard, moderately to highly fractured, slightly weathered; moderately soft and highly weathered between 4.3 and 5.2 feet; shattered to brecciated between 0 and 3.5 feet; scattered highly fractured to shattered zones from 0.1 to 2 feet thick throughout core; possible ½ inch wide shear zone dipping 60 degrees from the horizontal composed of decomposed granite between 27.0 and 27.3 feet; fracture surfaces are generally smooth and occasionally rust stained or calcite lined; fractures dip predominantly 40 to 90 degrees from the horizontal; ⅛ to ¼-inch wide seams of grout partially to completely fill approximately 50 percent of the fractures between 22.1 and 30 feet; ⅛-inch wide seams of grout completely fill fractures between 32.4 and 33.8 feet.

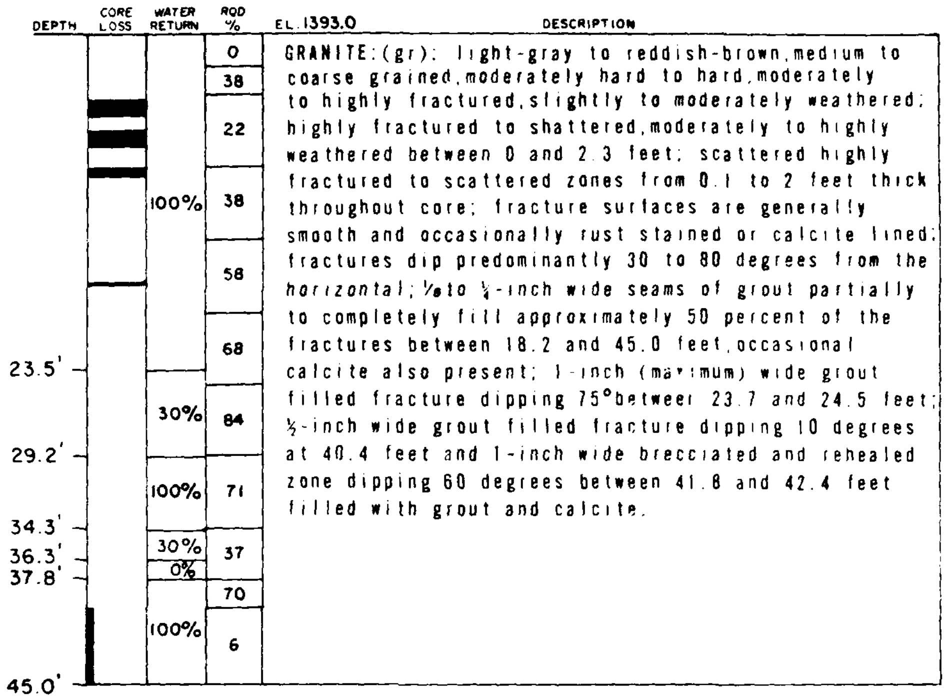
NOTES:

1. Total core recovery 88 percent.
2. See table 8, Foundation grouting summary, for pressure test and grouting data.

FIGURE 5. LOG OF EXPLORATORY CORE HOLE 102C.

103C

(STA. 13+29, 2.7' L)



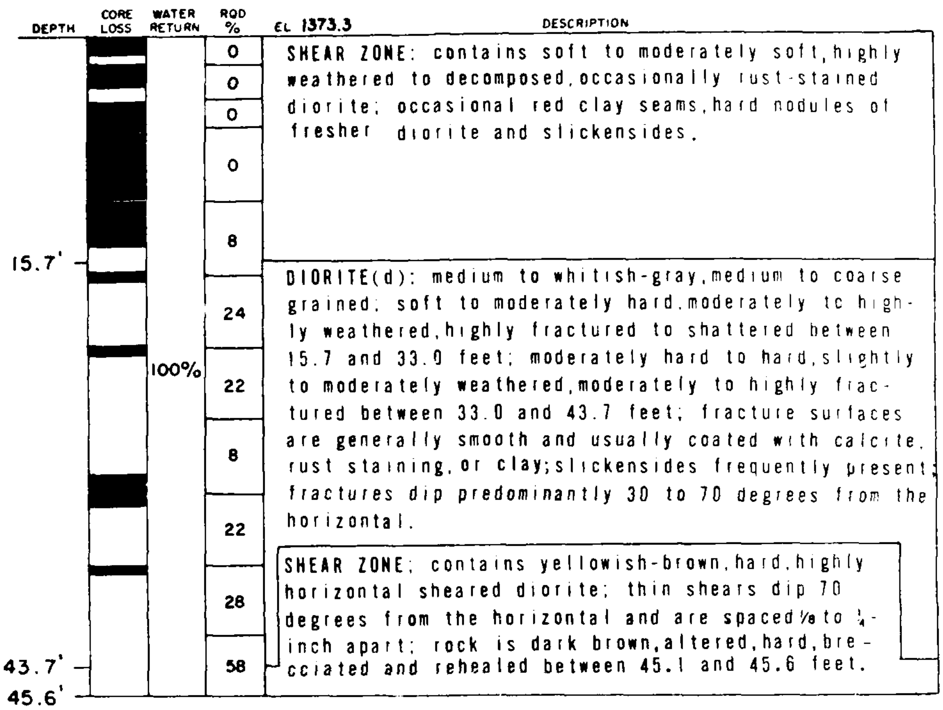
NOTES:

1. Total core recovery 93 percent.
2. See table 8, Foundation grouting summary, for pressure test and grouting data.

FIGURE 6. LOG OF EXPLORATORY CORE HOLE 103C

104C

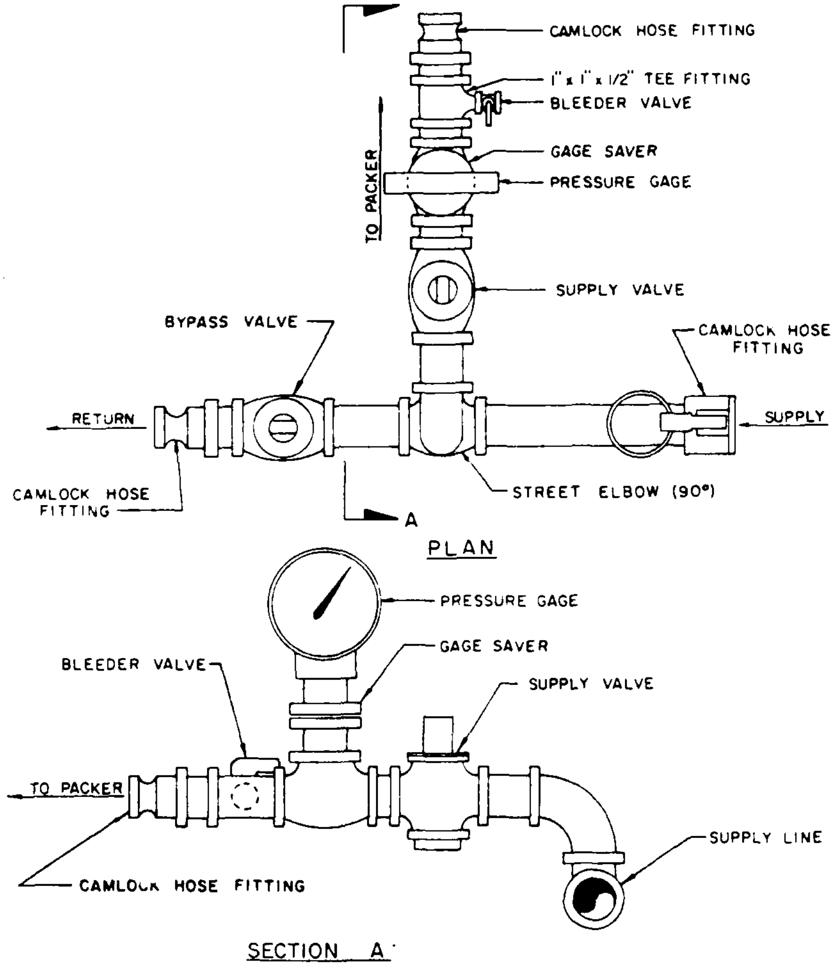
(STA. 17+49, 20' L)



NOTES:

1. Total core recovery 64 percent.
2. See table 8, Foundation grouting summary, for pressure test and grouting data.

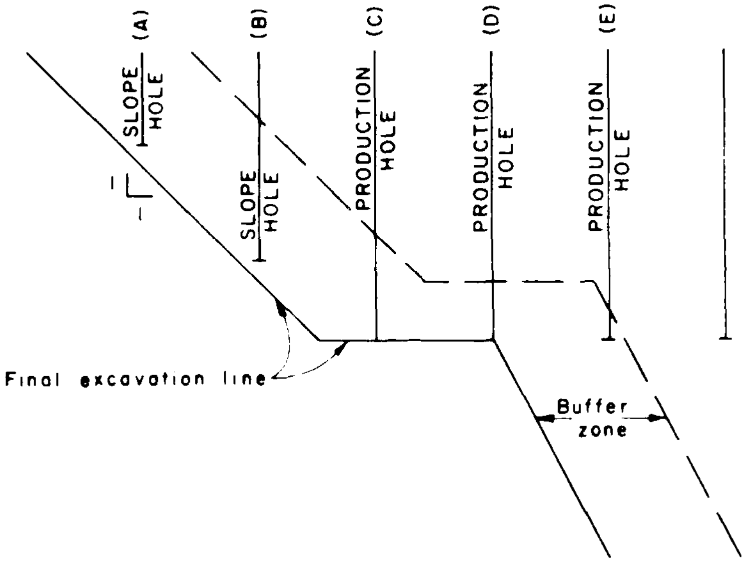
FIGURE 7. LOG OF EXPLORATORY CORE HOLE 104C



DIRECT GROUTING HEADER

NOT TO SCALE

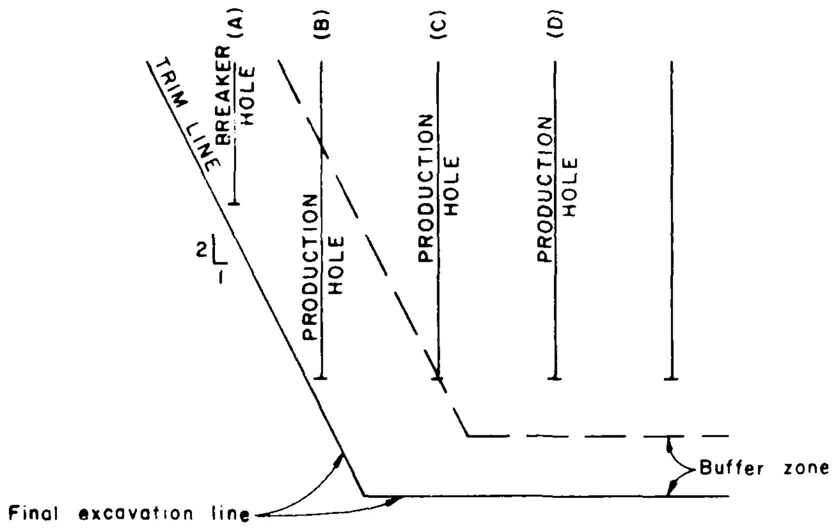
FIGURE 8.



STEP-DRILLING PLAN

1. Drilled 8'x8' pattern, hole depths varied from 6 to 20 feet.
2. Slope holes drilled to within 1 foot above final excavation line.
3. Poduction holes drilled down to bench elevation.
4. Holes loaded with Atlas ANFO with 1 stick of 1 3/4" x 8" Atlas Power Primer in bottom of each hole.
5. Shot delayed in following order (E through A) with millisecond delays.

FIGURE 9. APPROVED SPILLWAY BLASTING PLAN ABOVE BENCH



TRIM BLAST PLAN

1. Drilled 6'x6', 8'x8' or 9'x9' production hole pattern; hole depths averaged 22 feet.
2. Trim line holes drilled along final excavation lines on 4-foot centers.
3. Breaker hole spacing same as production hole spacing; however, the burden was generally 6 to 7 feet. Hole depths averaged 11 feet and drilled to within 1 foot above final excavation line.
4. Production and breaker holes loaded with ANFO with 1 stick of 1 3/4" x 8" Atlas Power Primer in bottom of each hole.
5. Trim line holes loaded with Atlas Kleen Kut powder.
6. Shot delayed in following order (D through A) with millisecond delays.
7. Invert buffer zone shot using plan similar to figure 9. Holes drilled on 6 x 6 foot pattern with 1 foot of subdrilling.

FIGURE 10. APPROVED SPILLWAY BLASTING PLAN BELOW BENCH

Photo 1. Upstream face of dam embankment as viewed from spillway excavation. 20 March 1985

Photo 2. View of dam embankment crest and upstream face looking toward right abutment. 20 March 1985

Photo 3. Landscaped downstream face of dam embankment as viewed from spillway access road. 20 March 1985

Photo 4. Landscaped downstream face of dam embankment and right abutment contact area as viewed from outlet channel. Note abutment staging platform excavation above dam crest. 18 April 1985

Photo 5. Upstream face of dam embankment and left abutment contact area. 18 April 1985

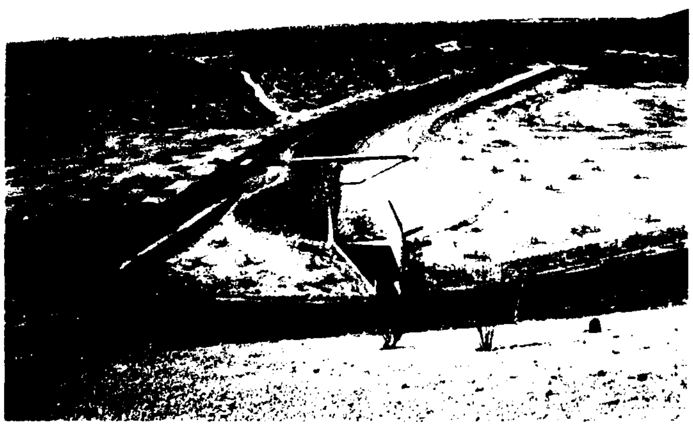


Photo 6. Looking downstream at main access road, energy dissipator, gaging station bridge, and outlet channel from crest of dam. 19 April 1985

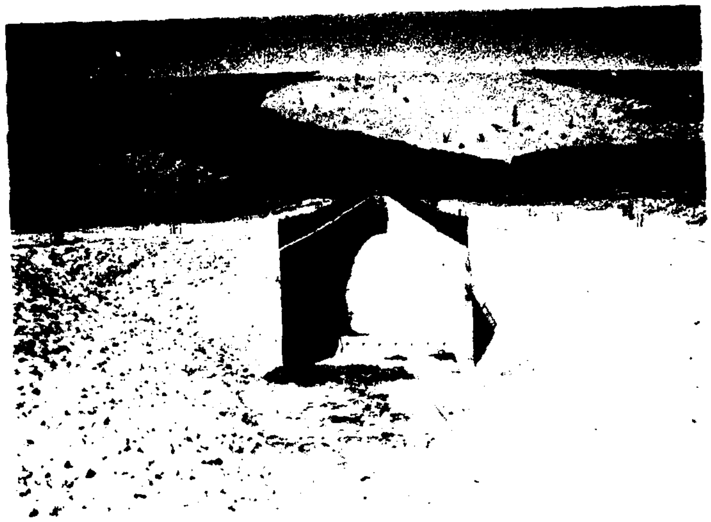


Photo 7. Looking upstream at energy dissipator and downstream face of dam embankment from gaging station bridge. 20 March 1985

Photo 8. Approach channel to intake structure as viewed from crest of dam. Note ditch to intercept slope runoff at right. 19 April 1985

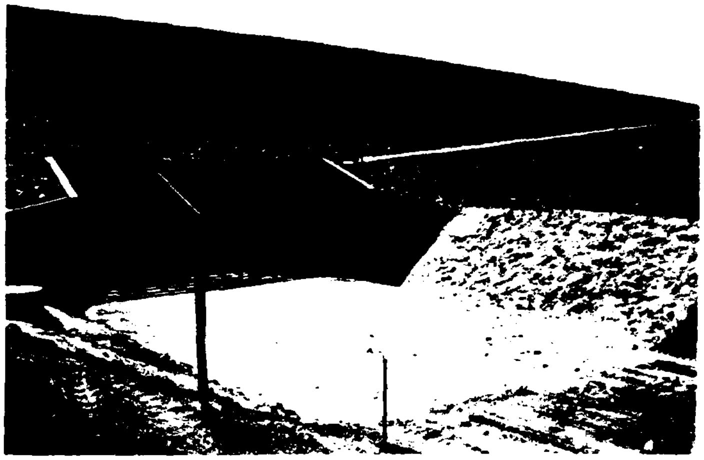


Photo 9. Intake structure and trash rack. Note grouted stone section of approach channel. 19 April 1985

Photo 10. Spillway excavation as viewed from upstream borrow area. Miscellaneous fill area in center of photo and right abutment of dam at left. 19 April 1985

Photo 11. View of completed spillway excavation looking upstream. 20 March 1985

Photo 12. Looking upstream at north wall of completed spillway excavation. Note concrete sill on lower slope. South wall bench at right. 20 March 1985

Photo 13. Crest and upstream face of dike No. 1 embankment as viewed from south abutment. Note grouted stone gutter along embankment toe. 20 March 1985

Photo 14. View of landscaped downstream face of dike No. 1 embankment looking toward south abutment. 20 March 1985

Photo 15. Downstream face of dike No. 2 embankment.
20 March 1985

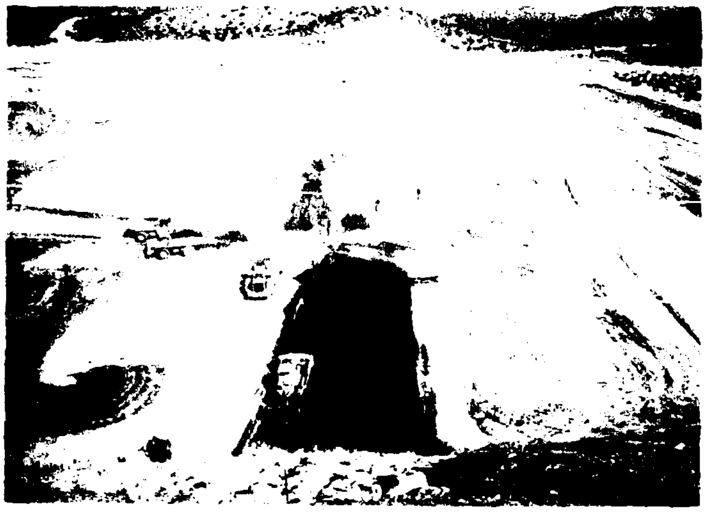


Photo 16. Stage III dam embankment construction as viewed from right abutment staging platform. Note zoned embankment, including downstream horizontal toe drain at right.
9 October 1984

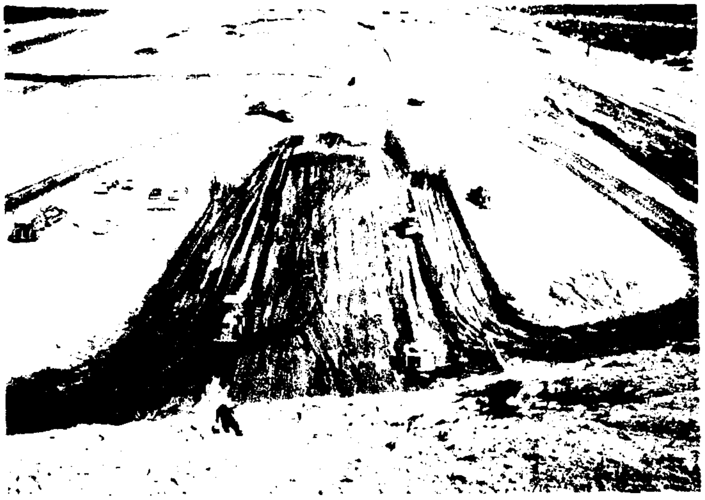


Photo 17. Stage I dam embankment fill placement. Note breach in Stage I diversion levee to allow equipment access. 28 February 1984

Photo 18. Upstream face of nearly completed Stage II dam embankment. Closure section will tie into sloping portion of embankment at right. 11 October 1984

Photo 19. Stage III dam embankment construction at approximately elevation 1420. Completed Stage II embankment with upstream stone protection at left. 25 October 1984

Photo 20. View of completed Stage II excavation to station 16+50. Note Stage I diversion levee and right abutment in background. 14 January 1984

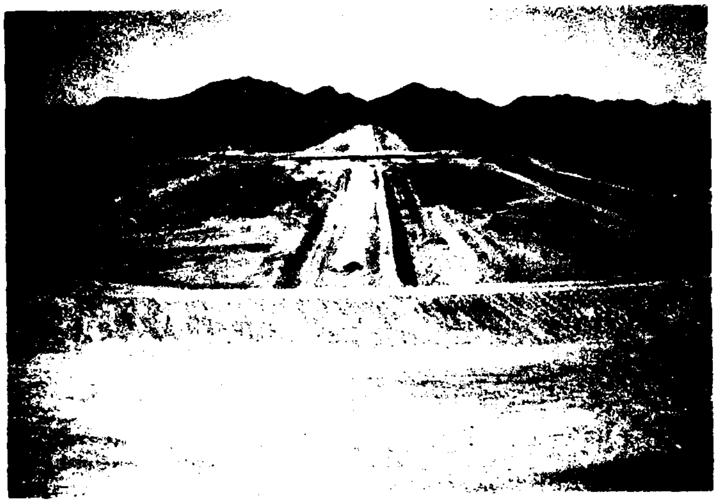


Photo 21. View of Stage II diversion levee protecting completed Stage II excavation to station 26+50. Note Stage I embankment protective cover in foreground. 31 March 1984

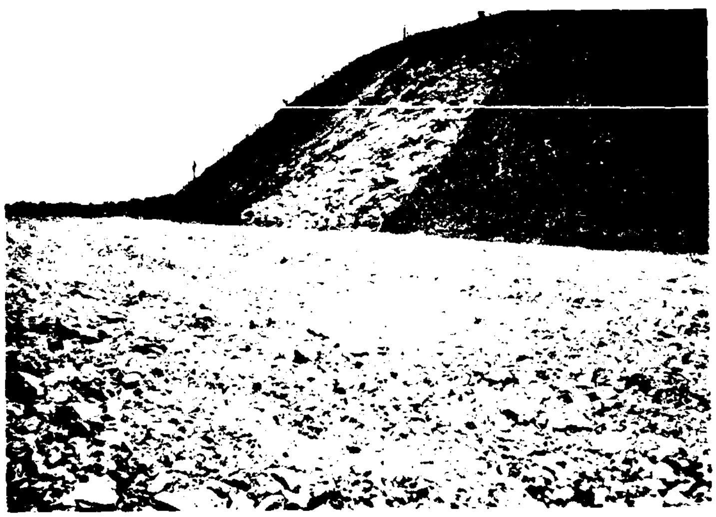


Photo 22. Protective cover of spillway rock over Stage I embankment fill. Right abutment in background. 31 March 1984

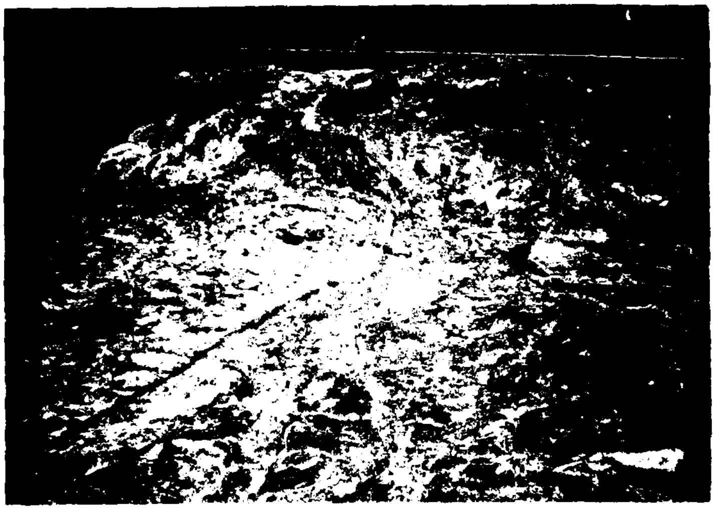


Photo 23. Looking east at core trench bedrock slope, stations 16+40 to 16+90. Note extensive shearing, particularly in upstream transition zone at left. 29 March 1984

Photo 24. Looking east at core trench excavation from station 19+30±. Note extensive scouring and water rounding of granitic bedrock surface. 29 March 1984

Photo 25. Northeast trending dike of intrusive igneous rock cutting highly fractured diorite; upstream of core trench station 14+00 near core-transition contact. 14 March 1984

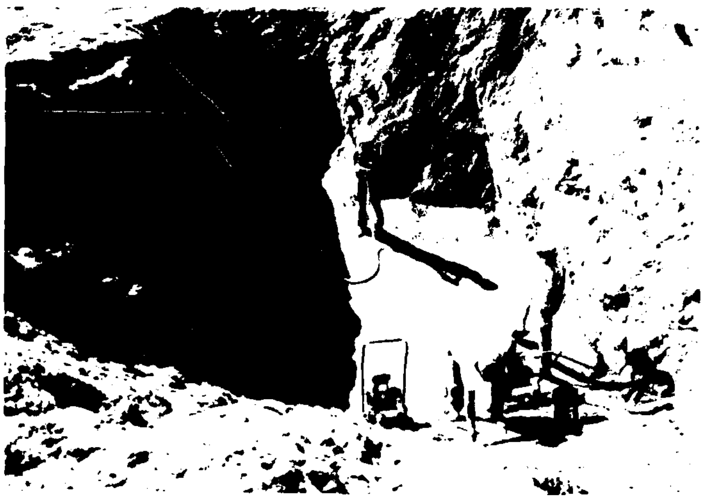


Photo 26. Backfilling foundation depression upstream of dam centerline between stations 14+85 and 15+25. 7 February 1984

Photo 27. Looking downslope from top of left abutment at major shear zone paralleling dam axis. Note shear zone confined mostly to upstream transition zone to right of paint stripe. 14 March 1984

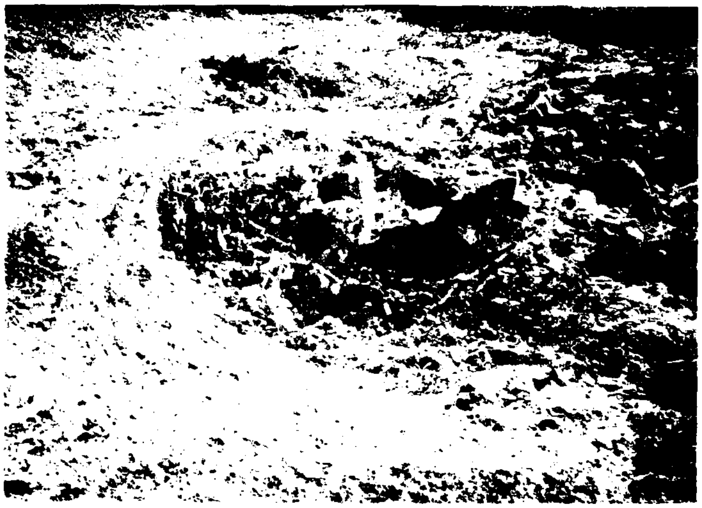


Photo 28. Major shear zone appears to wrap around lense of intensely fractured intrusive rock and blocky knob of diorite. 14 March 1984

Photo 29. Looking west at core trench excavation from top of slope at station 16+40. Note major shear zone near upstream edge of excavation at right and smaller individual shears which criss-cross remainder of foundation surface. Contact between granite and diorite (in foreground) near center of photo. 29 March 1984

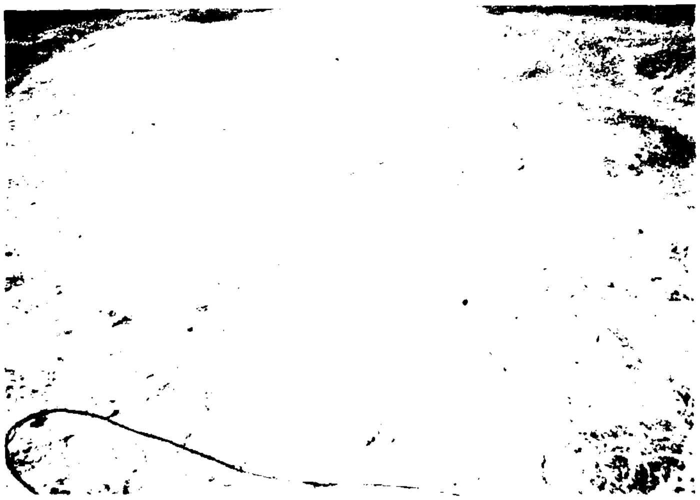


Photo 30. Typical example of extensive network of calcium carbonate healed fractures in diorite rock mass, vicinity of core trench station 14+70. 24 April 1984

Photo 31. Typical example of tight to mostly calcium carbonate healed blocky joint structures in granite rock mass, vicinity of core trench station 12+50.
14 March 1984

Photo 32. Typical example of strong subparallel to wedge shaped jointing in andesite bedrock, vicinity of core trench station 30+50. Joints generally strike N10°E, dip 90°.
11 February 1984

Photo 33. View of right abutment prior to start of Stage III construction. Note highly fractured, blocky nature of andesite rock mass and well-defined layering which dips obliquely upstream (toward lower right of photo). 26 September 1984

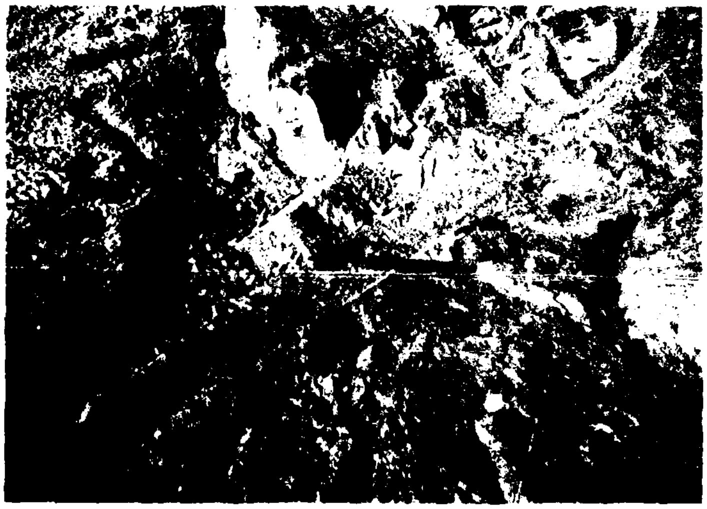


Photo 34. Joint (striking N50°W, dipping 55°SW) containing up to 6 inches of rehealed andesite breccia, core trench station 31+10. 11 February 1984

Photo 35. Stage II foundation excavation using push cats, scrapers, and D8 dozer. Note lighter colored areas of caliche or bedrock in upstream portion of dam foundation.
6 January 1984

Photo 36. View of completed Stage I core trench excavation and right abutment. Note backhoe trench in foreground.
8 Feburary 1984

Photo 37. Stage II core trench and left abutment after excavation and initial foundation preparation completed to station 16+50. 14 January 1984

Photo 38. D9H dozer ripping lower reach of right abutment access road. 28 October 1983

Photo 39. Right abutment "stripping" using Case 1150C dozer. 22 November 1983

Photo 40. Typical example of large roots and pockets of soil between andesite blocks at a depth of about one foot below the original right abutment surface. 6 December 1983

Photo 41. D8H dozer ripping lower right abutment surface.
2 December 1983

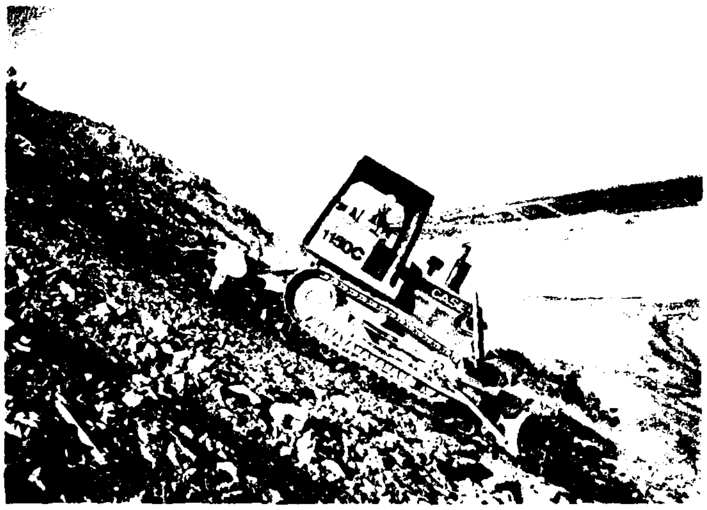


Photo 42. Case 1150C dozer ripping right abutment with
double shank ripper teeth; note double parted winch cable.
6 December 1983

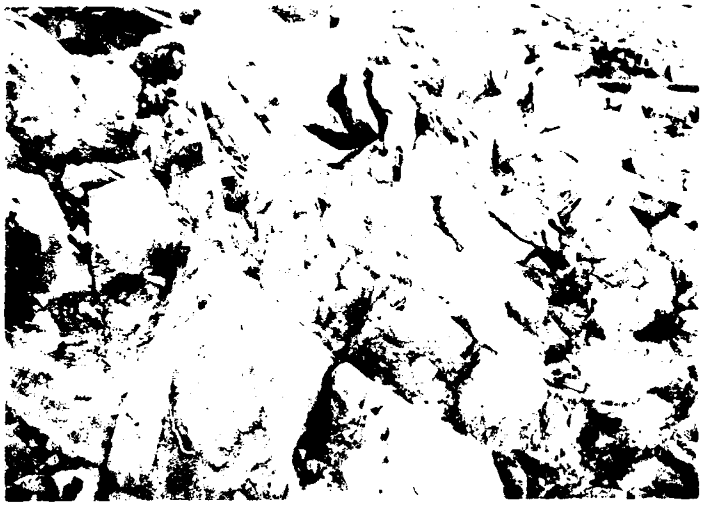


Photo 43. Typical example of excavated and blown right abutment surface prior to removing loose rock blocks. Note distinct joints. 8 December 1983

Photo 44. Stage I core trench excavation using push cat and scraper. Note D8 dozer using rippers and blade to excavate base of right abutment. 16 December 1983

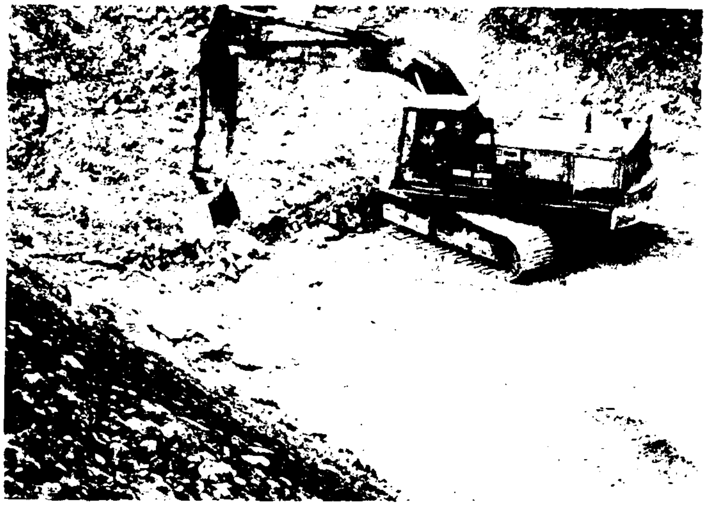


Photo 45. Excavating andesite bedrock in Stage I core trench using Catepillar 235 excavator, vicinity station 31+10. 22 December 1983

Photo 46. Excavator widening Stage I core trench. January 1984

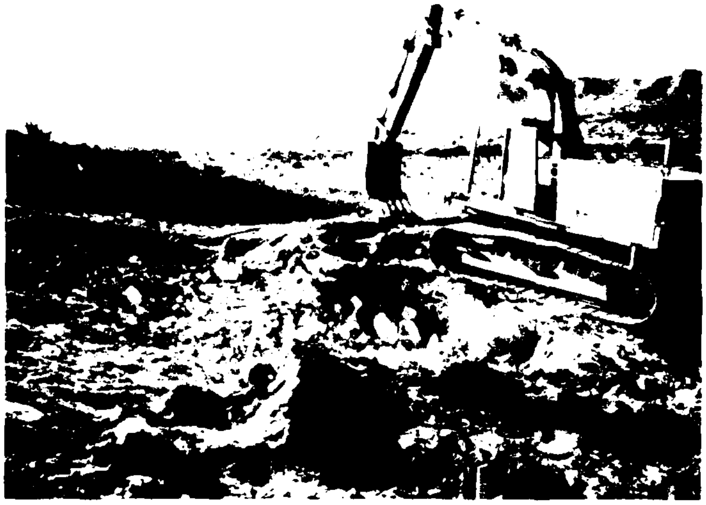


Photo 47. Left abutment excavation using 235 excavator.
January 1984

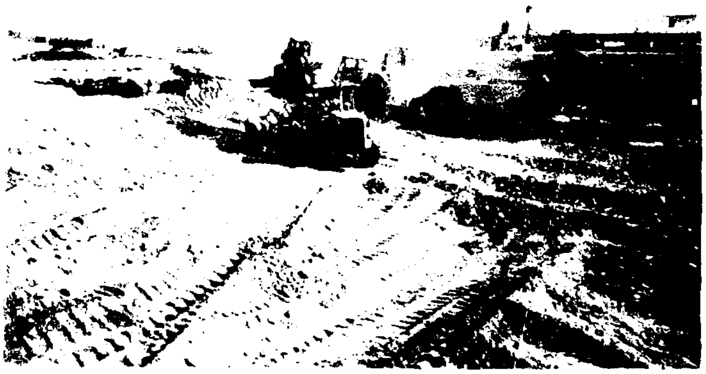


Photo 48. Stage II core trench excavation using push cats and scrapers. Note front end loader pushing overburden away from bedrock slope near station 16+50 for scraper removal.
9 March 1984

Photo 49. Laborers tied off to safety lines while cleaning steep right abutment surface. 29 November 1983

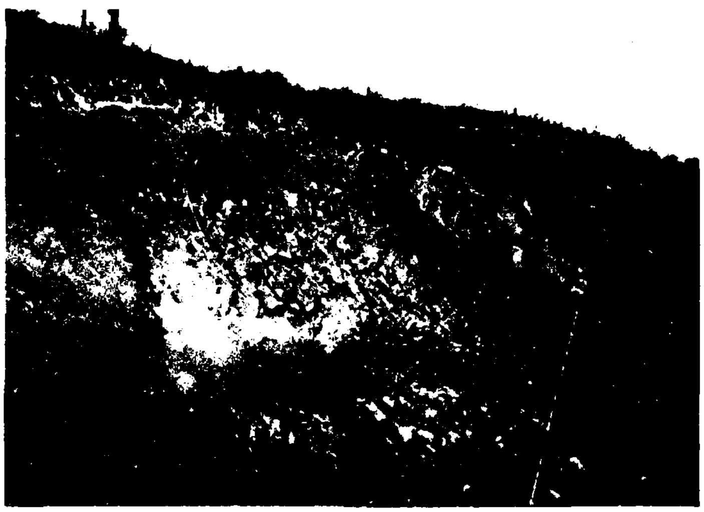


Photo 50. Surface preparation of right abutment using both air blasting and hand labor. December 1983

Photo 51. Foundation preparation of right abutment surface using high pressure air blasting. Note excavator working on south side of Stage I core trench. 21 December 1983

Photo 52. Foundation preparation of Stage I core trench surface using backhoe and hand labor, vicinity station 30+00. Note darker colored rock is agglomerate cap. 6 January 1984

Photo 53. Pump truck with extendable boom and hose used for dental concrete placement in Stage I core trench. 10 February 1984

Photo 54. Placing and vibrating dental concrete in Stage I core trench. 10 February 1984

Photo 55. Stage I core trench, vicinity station 31+50, after completion of required surface treatment. Note large areas of dental concrete. 11 February 1984

Photo 56. Placing grout slurry mix by hand in prewetted crack between rock blocks, vicinity station 31+80. 10 February 1984

Photo 57. Beginning Stage III right abutment surface preparation. Laborers permitted to work only short reaches without safety lines. Note backhoe removing Stage I protective cover down to elevation 1380 near base of slope. 29 September 1984

Photo 58. Closeup of localized 3 foot deep cavity created during surface preparation of right abutment; 20 feet right of dam centerline, approximate station 31+96. 2 October 1984

Photo 59. Placing dental concrete on right abutment slope using concrete bucket. 2 October 1984

Photo 60. Closeup of grout slurry application on right abutment. Slurry placed in 1 inch wide by 6 inch deep open joints between andesite blocks. 9 November 1984

Photo 61. Right abutment between elevations 1393 and 1405 after surface treatment. 12 October 1984

Photo 62. Small Case backhoe assisting excavator during left abutment excavation. Note rock dust generated from air blasting of excavated abutment surface. 10 January 1984

Photo 63. Initial foundation preparation of left abutment surface using low pressure air blasting. Note 235 excavator on left. 6 January 1984

Photo 64. Foundation preparation in Stage II core trench using backhoes and shovels; vicinity station 17+00. 12 March 1984

Photo 65. Air cleaning core trench surface at station 20+80 using high pressure bull hose. 16 March 1984

Photo 66. Stage II core trench foundation preparation. Laborers removing alluvial materials loosened by backhoe from bedrock scour channels. 26 March 1984

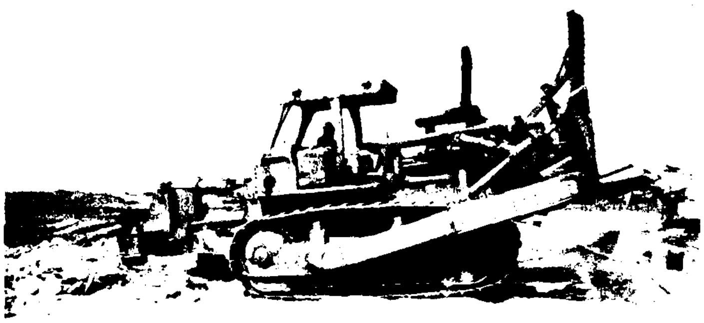


Photo 67. D9H dozer with double shank ripping calichified granite cap at the upstream edge of the core trench, vicinity station 16+30. 12 March 1984

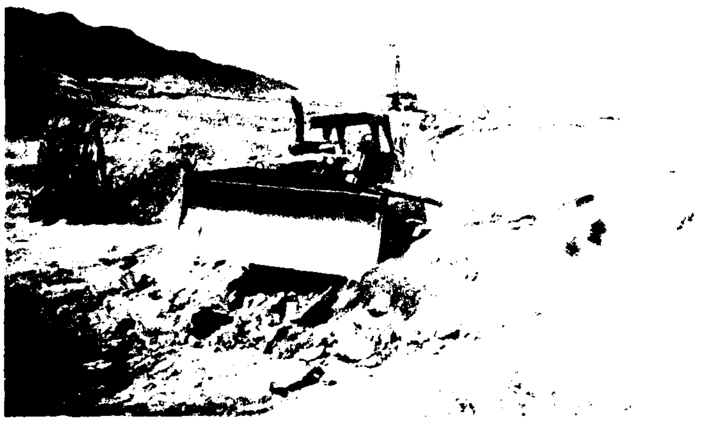


Photo 68. D9H dozer using slopeboard to flatten bedrock slope near station 16+50 to 1:1 and remove soft weathered bedrock from foundation surface. 12 March 1984

Photo 69. Left abutment between stations 11+48 and 12+24 after dental concrete placement. 17 May 1984

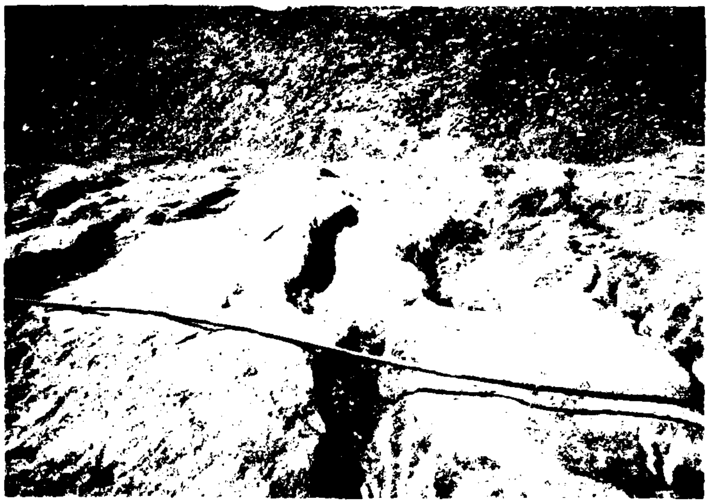


Photo 70. Typical example of scour channels incised in granite bedrock, station 19+65. View looking upstream. 29 March 1984

Photo 71. Stage II core trench scour channels between stations 19+10 and 18+10 backfilled with dental concrete. 31 March 1984

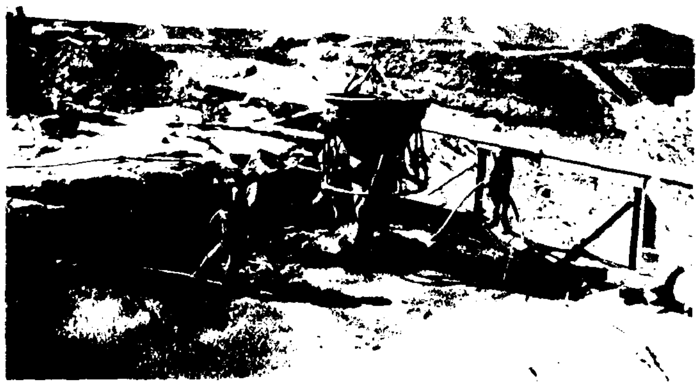


Photo 72. Backfilling preconstruction test trench between stations 16+00 and 16+50, downstream side of core trench, with dental concrete. Note sloping bulkhead constructed across open end of trench to contain concrete. 30 March 1984

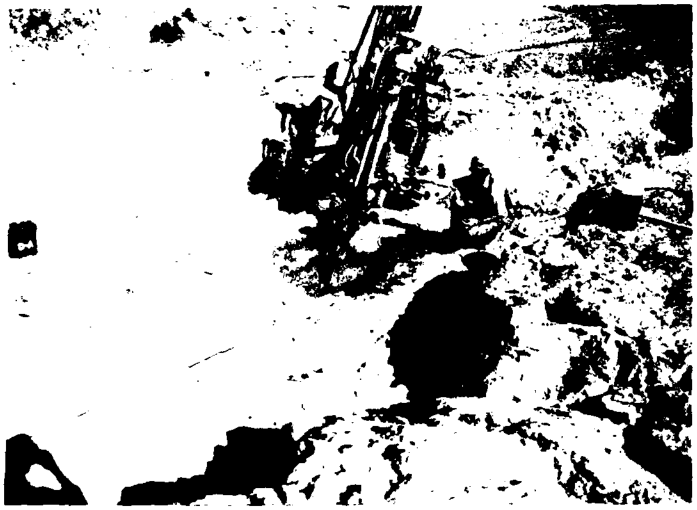


Photo 73. Drilling first zone of primary grout hole 30+00P using air track percussion drill. Air-water mixture used as circulating medium. 10 January 1984

Photo 74. Air track drilling first zone oi secondary grout hole 32+34S on right abutment slope. Drill pulled up slope using winch cable system. 6 March 1984

Photo 75. View of upstream end of intake structure foundation looking east. Note left side wing wall steps in background and shear zone extending across bottom of excavation. 6 February 1984

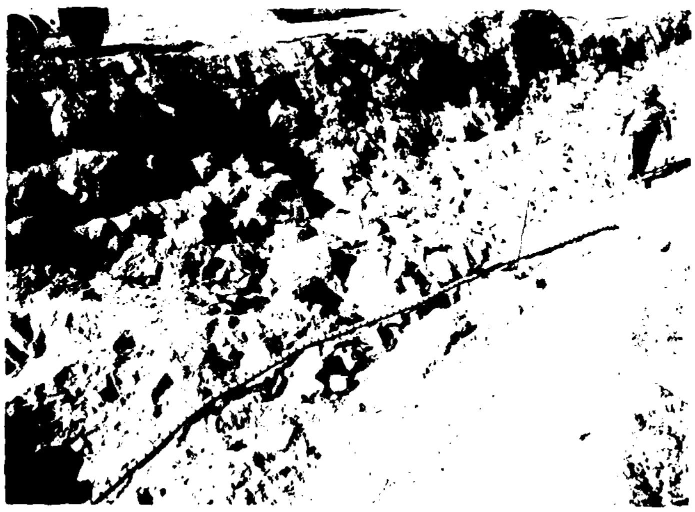


Photo 76. Outlet works conduit excavation between stations 17+60 and 17+40. Note large angular granite blocks on floor of trench. 23 January 1984

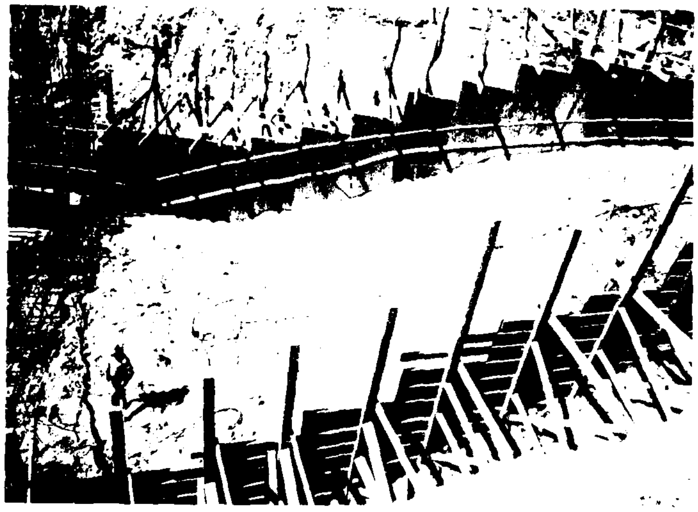


Photo 77. Overexcavated energy dissipator slope. Note chamfer lines on bulkhead. Bottom line is top of dental concrete or "B" line elevation while top line is invert of structural concrete. 19 March 1984

Photo 78. 235 excavator scraping off loosened rock from surface of outlet conduit trench. Note relatively level surface (where laborers are walking). 16 January 1984

Photo 79. D9 dozer ripping high spot at station 19+70 in outlet conduit trench excavation. 17 January 1984

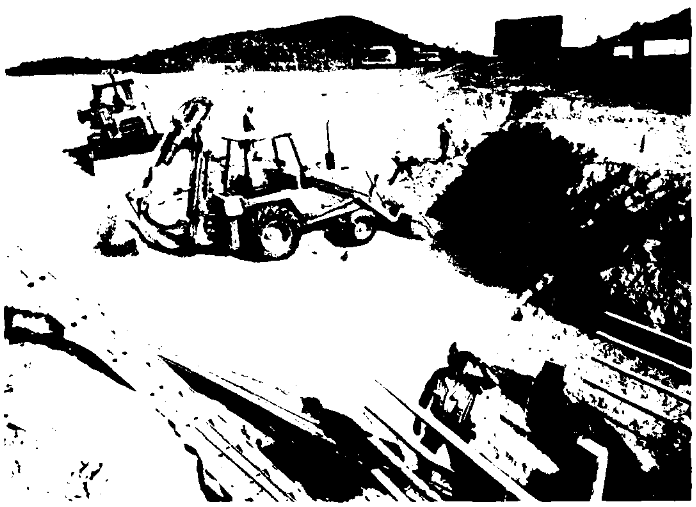


Photo 80. Intake structure foundation pr^paration. Note laborer jackhammering high spots on left (east) side wing wall steps down to grade. 1 February 1984

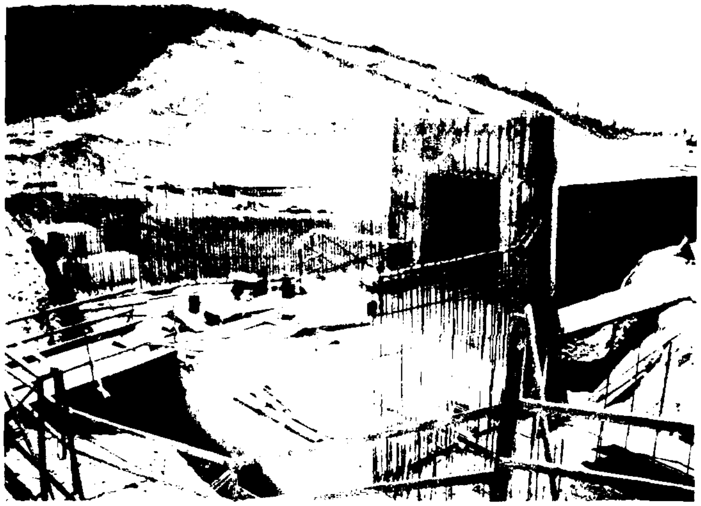


Photo 81. Partially completed outlet works intake structure. Note overexcavated area upstream of invert concrete slab. 26 March 1984

Photo 82. View of completed intake structure looking west. Note lean concrete backfill areas adjacent to structure. 11 June 1984

Photo 83. View of outlet works excavation looking upstream. Hard granitic bedrock exposed by excavator within energy dissipator in foreground. 18 January 1984

Photo 84. Foundation preparation of outlet conduit excavation using backhoe and hand labor. 18 January 1984

Photo 85. Air cleaning energy dissipator foundation with very low pressure air hose to avoid degradation of granite bedrock surface. Note hard granite blocks in decomposed granite matrix. 15 February 1984

Photo 86. Placing leveling slab of dental concrete in outlet conduit excavation. Wooden screed bar used for leveling surface of slab. 24 January 1984

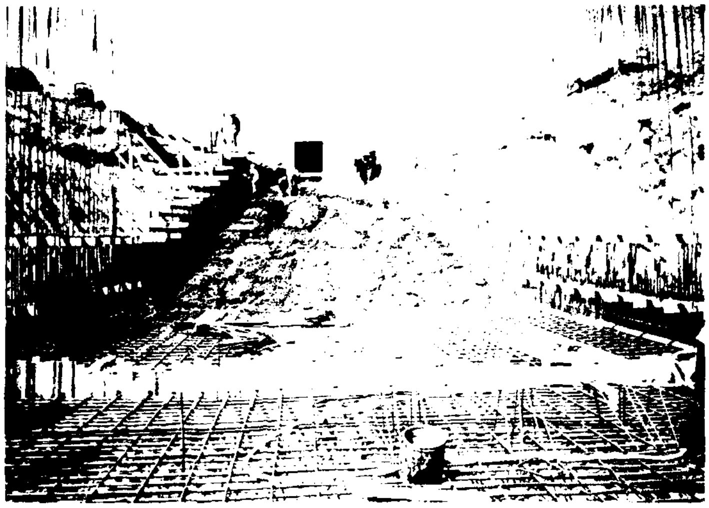


Photo 87. Looking upstream at dental concrete placement on energy dissipator slope. 20 March 1984

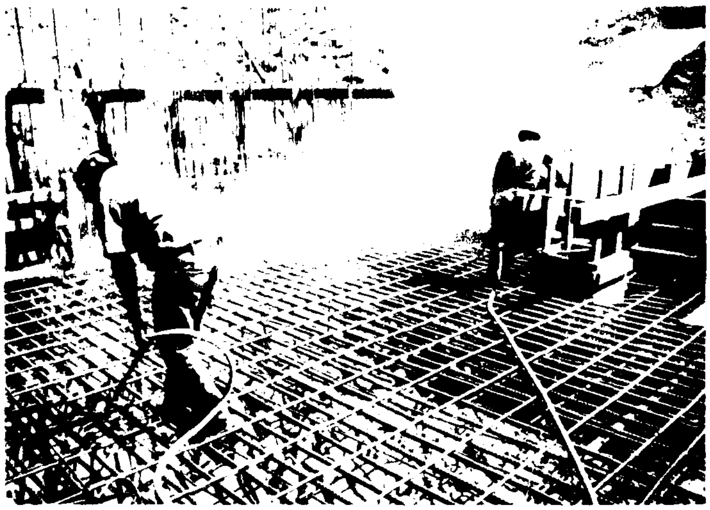


Photo 88. Final cleanup of energy dissipator foundation by low pressure air blasting. 20 March 1984

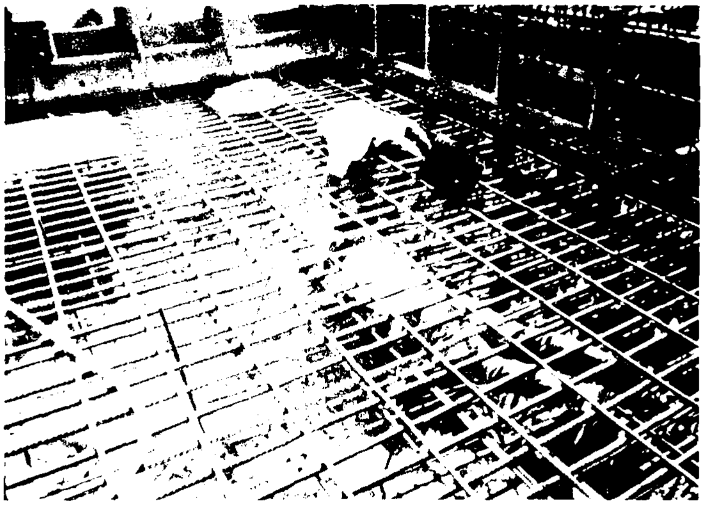


Photo 89. Hand picking loose rock from foundation surface underneath energy dissipator rebar lattice. 20 March 1984

Photo 90. Outline of concrete plug area on east side of outlet conduit. Note most of conduit projects above adjacent foundation surface due to greater depth of core trench excavation. 24 April 1984

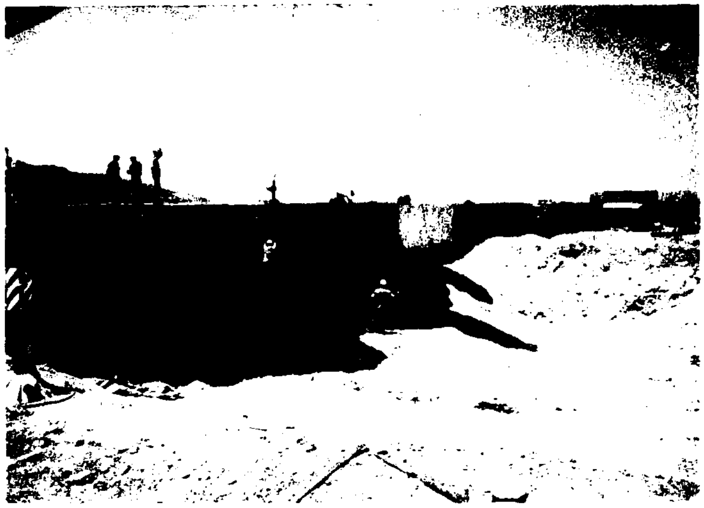


Photo 91. Concrete plug placement on east side of conduit using concrete bucket. Note laborer on top of box vibrating concrete near contact with conduit wall. 25 April 1984

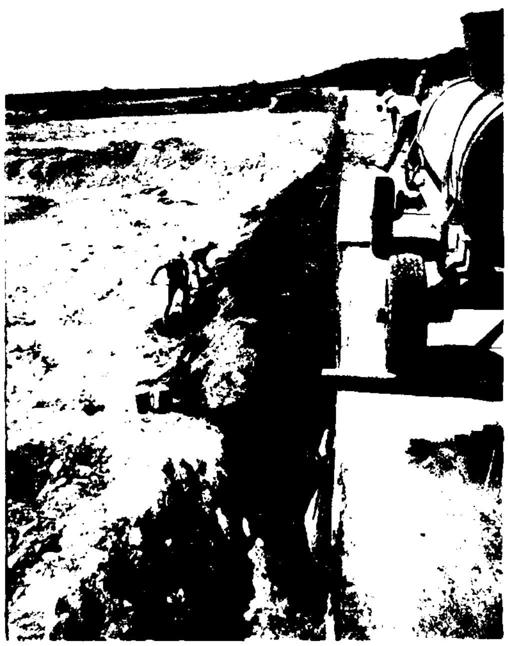


Photo 92. Concrete plug on west side of outlet conduit. Note upper portion of plug is slightly steeper than 1:1 to eliminate "feather" edges against the top of the conduit. 25 April 1984

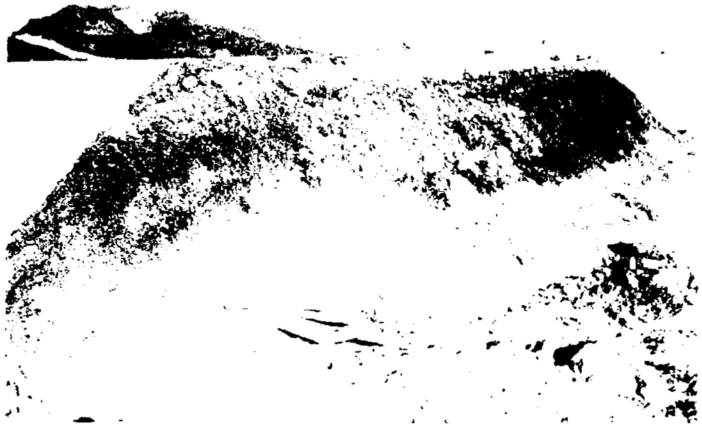


Photo 93. Completed concrete plug on east side of conduit. Concrete spills outside plug limits were subsequently removed. 26 April 1984

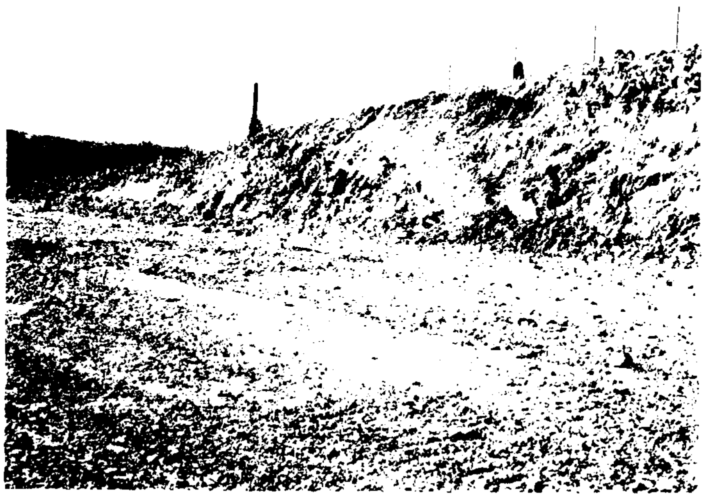


Photo 94. View of downstream north wall of completed spillway excavation showing layered volcanic sequence which dips 30°NE. Note massive flow breccia separating fractured andesite (on right) and tuff units. Alluvium exposed near edge of photo on left. 29 November 1984

Photo 95. Typical example of strongly jointed andesite flow exposed in north wall of spillway excavation, vicinity station 17+00. Note variable joint patterns and darker colored "zone of weathering" near surface of flow. 20 March 1985

Photo 96. Strong subparallel to wedge shaped joint pattern (striking N75°E, dipping 45°NW) exposed in north wall of spillway excavation below bench, vicinity station 15+50. 20 March 1985

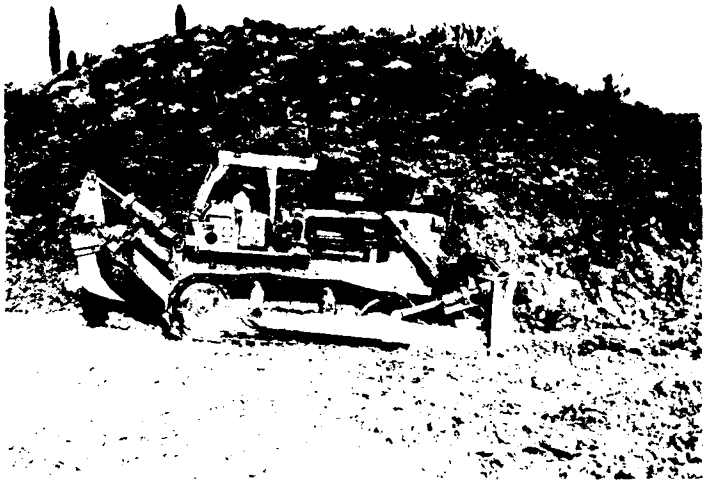


Photo 97. Mechanical excavation in spillway using D9H dozer with double shank. Note slopeboard used for slope trimming. 27 October 1983

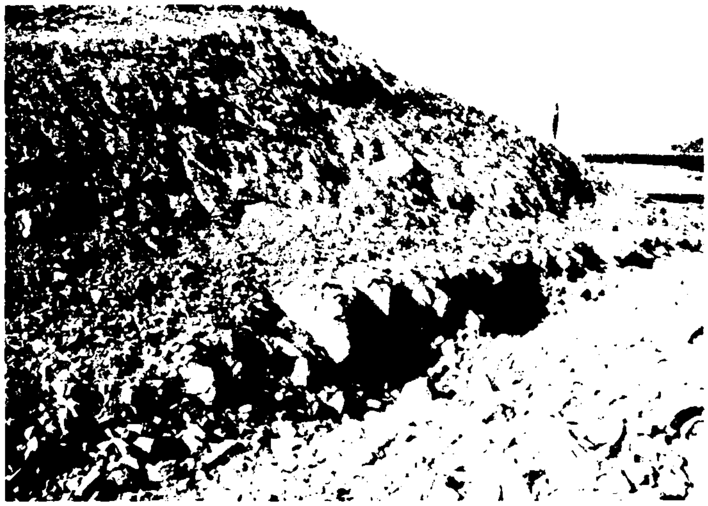


Photo 98. North side of spillway excavation after production shot No. 9 down to bench excavation. Note 1 to 2-foot buffer of rock left by step-drilling blasting technique which was subsequently removed to grade using dozer slopeboard. 9 March 1984

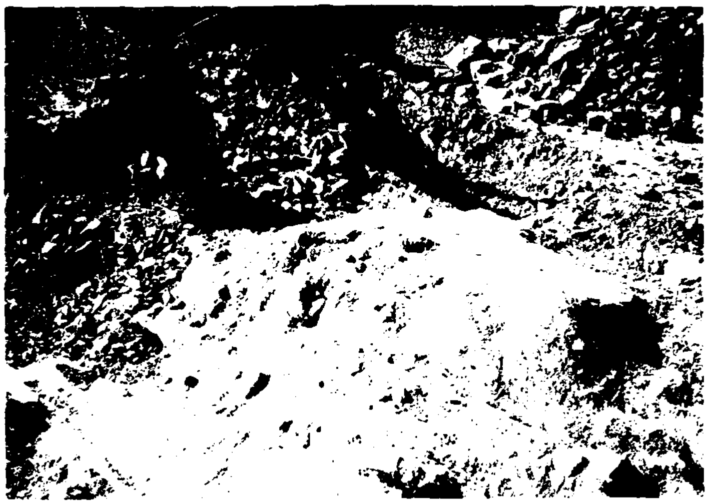


Photo 99. Spillway test area excavated and cleaned tc allow inspection and evaluation of proposed horizontal buffer zone blasting technique. Results unacceptable-note large 6 x 6-foot rock block virtuallv unaffected by blast. 6 September 1984

Photo 100. Scraper hauling shot rock from spillway for Stage II diversion levee slope protection. Dozer with slopeboard used for slope trimming. 22 March 1984

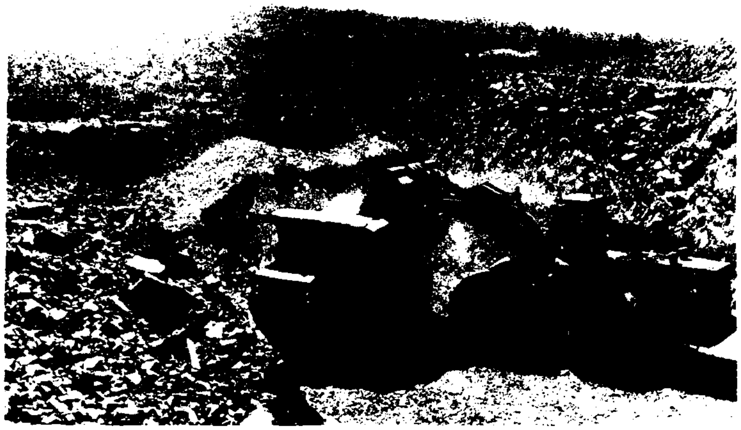


Photo 101. Excavating production shot No. 14 in spillway using front end loader and rock truck. 5 September 1984

Photo 102. Outline of sill section, south wall of spillway excavation. Note irregular slope and highly fractured andesite rock mass. 31 October 1984

Photo 103. Excavating spillway sill at station 17+50 using a Gradall with hydraulic ram attachment to break up the rock, and a Case backhoe to excavate the broken rock.
28 November 1984

Photo 104. Excavated sill section, north wall of spillway. Note limited over break in fractured andesite rock mass. 29 November 1984

Photo 105. Laborers cleaning loose rock from invert section of spillway sill. Variable bedrock structure resulted in wider and slightly deeper excavation than originally designed. 29 November 1984

Photo 106. Completed concrete sill section, north wall of spillway. Sill extends up to bench level. Note sloped invert section projecting above bedrock near bottom of photo. 20 March 1985

Photo 107. South wall of completed spillway excavation, vicinity station 16+00. Note variable joint orientations and spacings, including prominent dip-slope joint planes. 18 April 1985

Photo 108. Slope failure along arcuate dip-slope joint plane in spillway south wall at station 15+75 has reduced bench width to only 7 feet. 20 March 1985

Photo 109. Dike No. 1 exploration trench, station 81+25. Note coherent andesite blocks on both sides of photo with calichified andesite breccia toward center.
22 January 1984

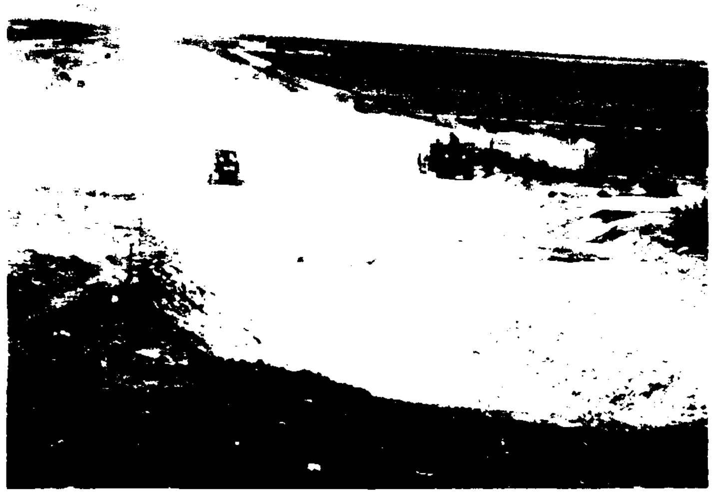


Photo 110. View of dike No. 1 exploration trench excavation looking north from south abutment. January 1984

Photo 111. View of dike No. 1 completed exploration trench excavation looking south from station 13+00. Note widespread caliche. 25 February 1984

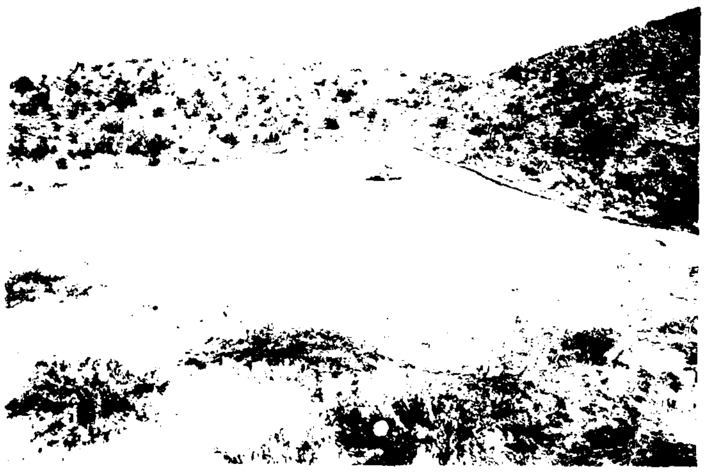
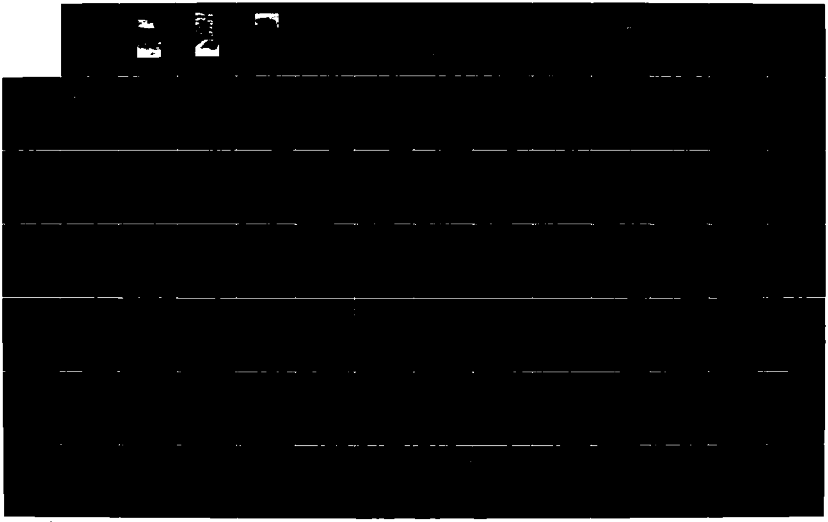


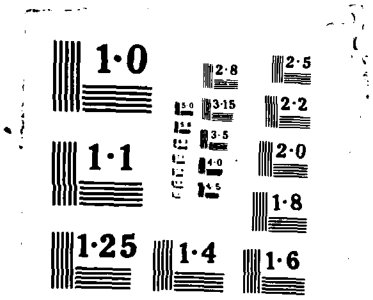
Photo 112. Dike No. [illegible] foundation after stripping. 6 February 1984

AD-A168 748 NEW RIVER DAM FOUNDATION REPORT GILA RIVER BASIN: PHOENIX ARIZONA AND VICINITY (INCLUDING NEW RIVER) (U) ARMY ENGINEER DISTRICT LOS ANGELES CA OCT 85

3/4

UNCLASSIFIED F/G 13/2 NL




1·0
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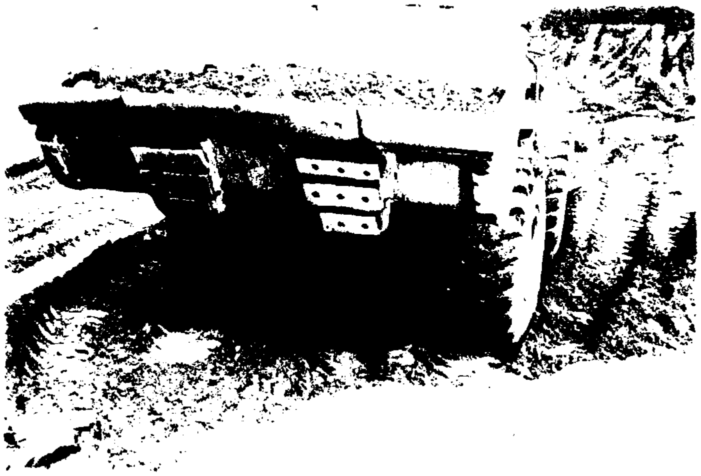


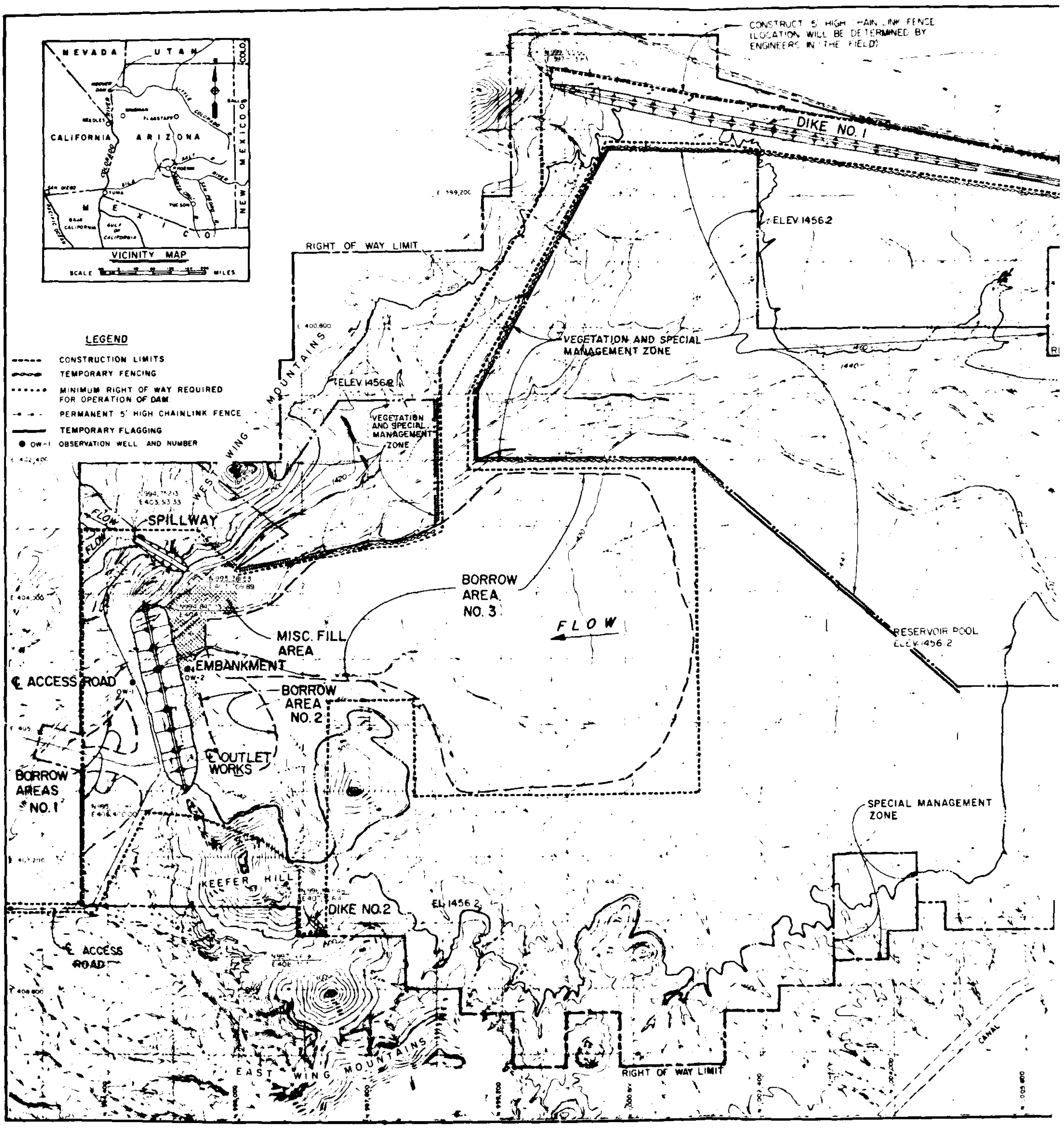
Photo 113. Front end loader with full bucket compacting initial lift of core material against treated bedrock surface in Stage I core trench, vicinity station 30+70. 13 February 1984

Photo 114. Front end loader compacting core material against bedrock slope in Stage II core trench, vicinity station 16+50. Note dozer using blade to spread core material over foundation surface. 2 April 1984

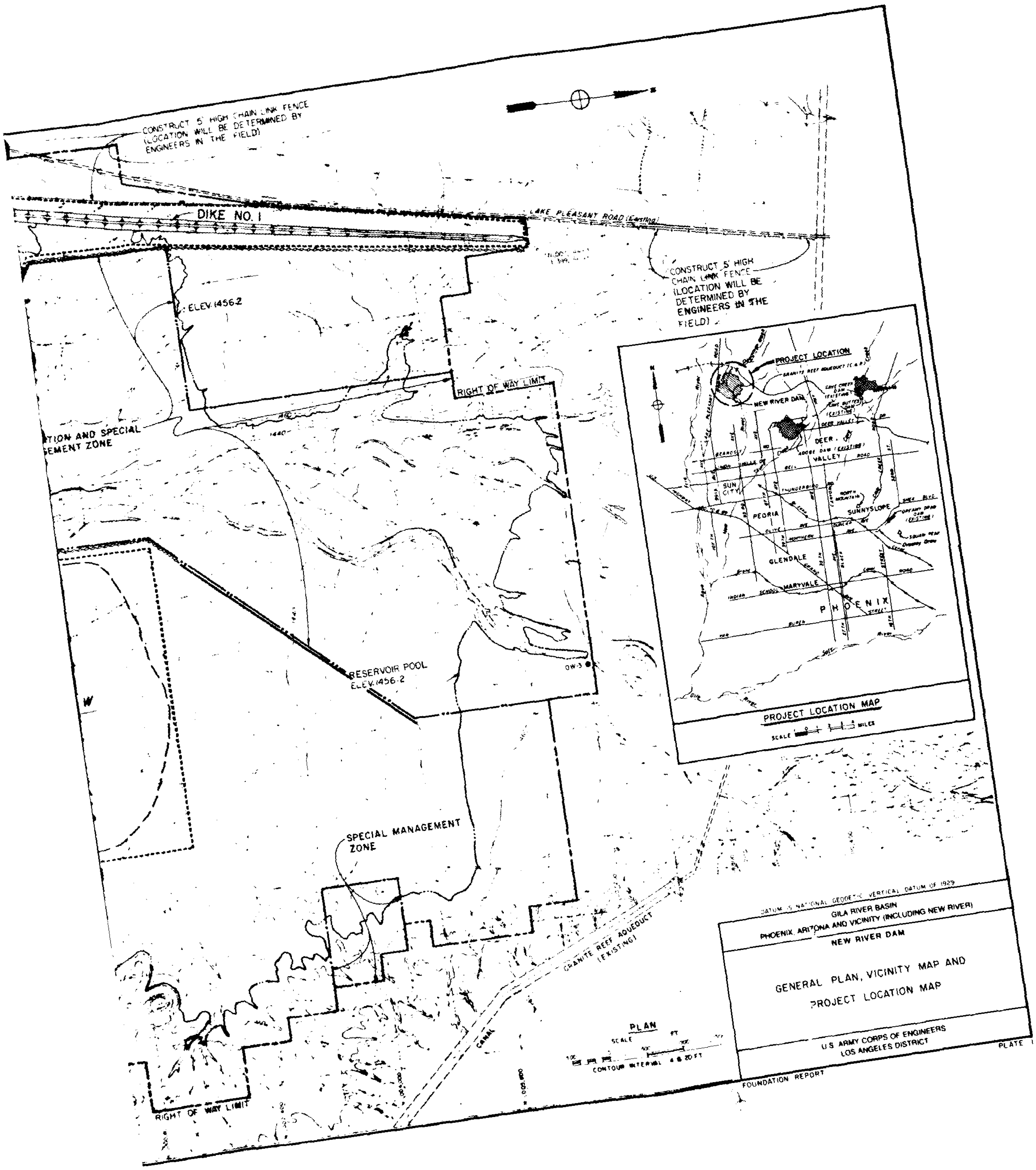
Photo 115. Typical example of rock-core contact on left abutment (approximate elevation 1447). Wheel rolling core material prevented damage to highly fractured granitic bedrock surface by tamping roller. 5 July 1984

Photo 116. Five foot high ramp of core material against right abutment slope. 22 October 1984

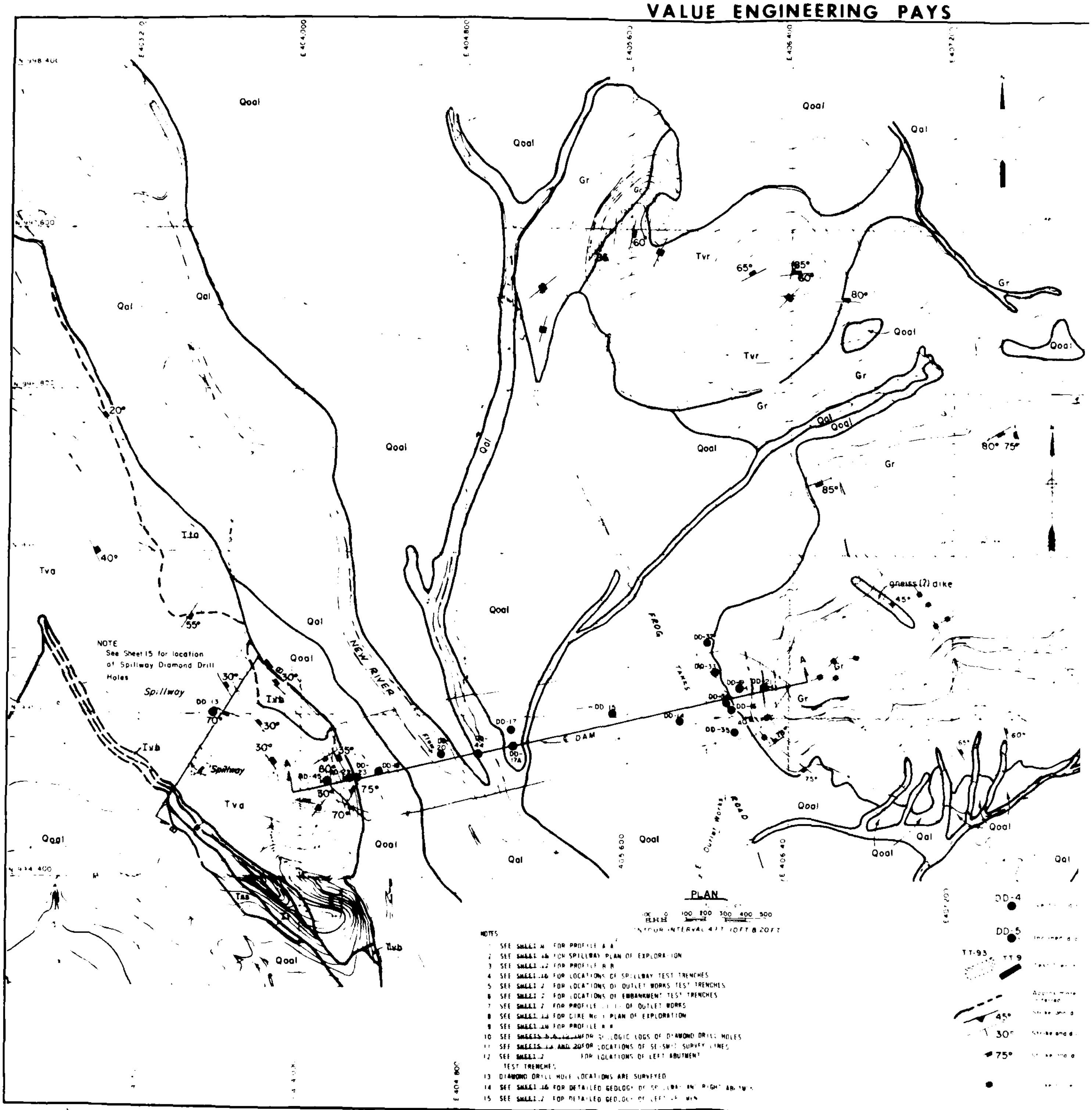
Photo 117. Slope failure in intensely fractured platy andesite, north wall of spillway excavation, vicinity station 14+00. 18 April 1985



NEVADA
UTAH
COLO.
CALIFORNIA
ARIZONA
NEW MEXICO
MEXICO
VICINITY MAP
SCALE
MILES
LEGEND
CONSTRUCTION LIMITS
TEMPORARY FENCING
MINIMUM RIGHT OF WAY REQUIRED FOR OPERATION OF DAM
PERMANENT 5' HIGH CHAINLINK FENCE
TEMPORARY FLAGGING
OW-1 OBSERVATION WELL AND NUMBER
CONSTRUCT 5' HIGH CHAIN LINK FENCE (LOCATION WILL BE DETERMINED BY ENGINEERS IN THE FIELD)
DIKE NO. 1
ELEV 1456.2
RIGHT OF WAY LIMIT
VEGETATION AND SPECIAL MANAGEMENT ZONE
WEST WING MOUNTAINS
SPILLWAY
FLOW
BORROW AREA NO. 3
MISC. FILL AREA
EMBANKMENT
℄ ACCESS ROAD
BORROW AREA NO. 2
℄ OUTLET WORKS
BORROW AREAS NO. 1
RESERVOIR POOL ELEV 1456.2
SPECIAL MANAGEMENT ZONE
KEEFER HILL
DIKE NO. 2
EL 1456.2
EAST WING MOUNTAINS
CANAL

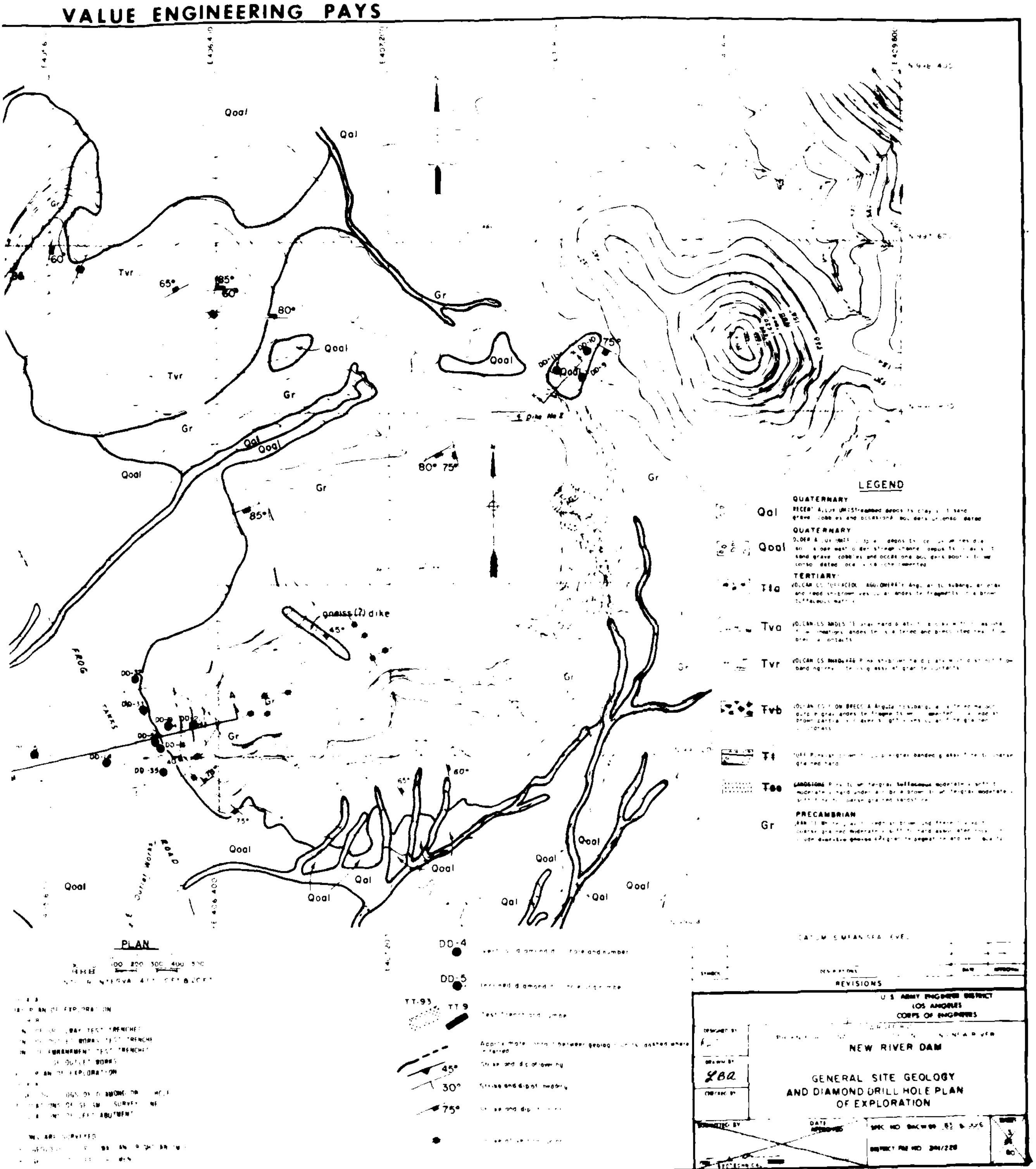

CONSTRUCT 5' HIGH CHAIN LINK FENCE (LOCATION WILL BE DETERMINED BY ENGINEERS IN THE FIELD)
DIKE NO. 1
LAKE PLEASANT ROAD (Existing)
CONSTRUCT 5' HIGH CHAIN LINK FENCE (LOCATION WILL BE DETERMINED BY ENGINEERS IN THE FIELD)
ELEV 1456.2
RIGHT OF WAY LIMIT
TION AND SPECIAL
GEMENT ZONE
RESERVOIR POOL ELEV 1456.2
OW-3
SPECIAL MANAGEMENT ZONE
GRANITE REEF AQUEDUCT (EXISTING)
CANAL
RIGHT OF WAY LIMIT
PLAN
CONTOUR INTERVAL 4 & 20 FT
PROJECT LOCATION
NEW RIVER DAM
DEER VALLEY
SUN CITY
PEORIA
SUNNYSLOPE
GLENDALE
MARYVALE
PHOENIX
PROJECT LOCATION MAP
DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929
GILA RIVER BASIN
PHOENIX, ARIZONA AND VICINITY (INCLUDING NEW RIVER)
NEW RIVER DAM
GENERAL PLAN, VICINITY MAP AND PROJECT LOCATION MAP
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT
PLATE 1
FOUNDATION REPORT

VALUE ENGINEERING PAYS



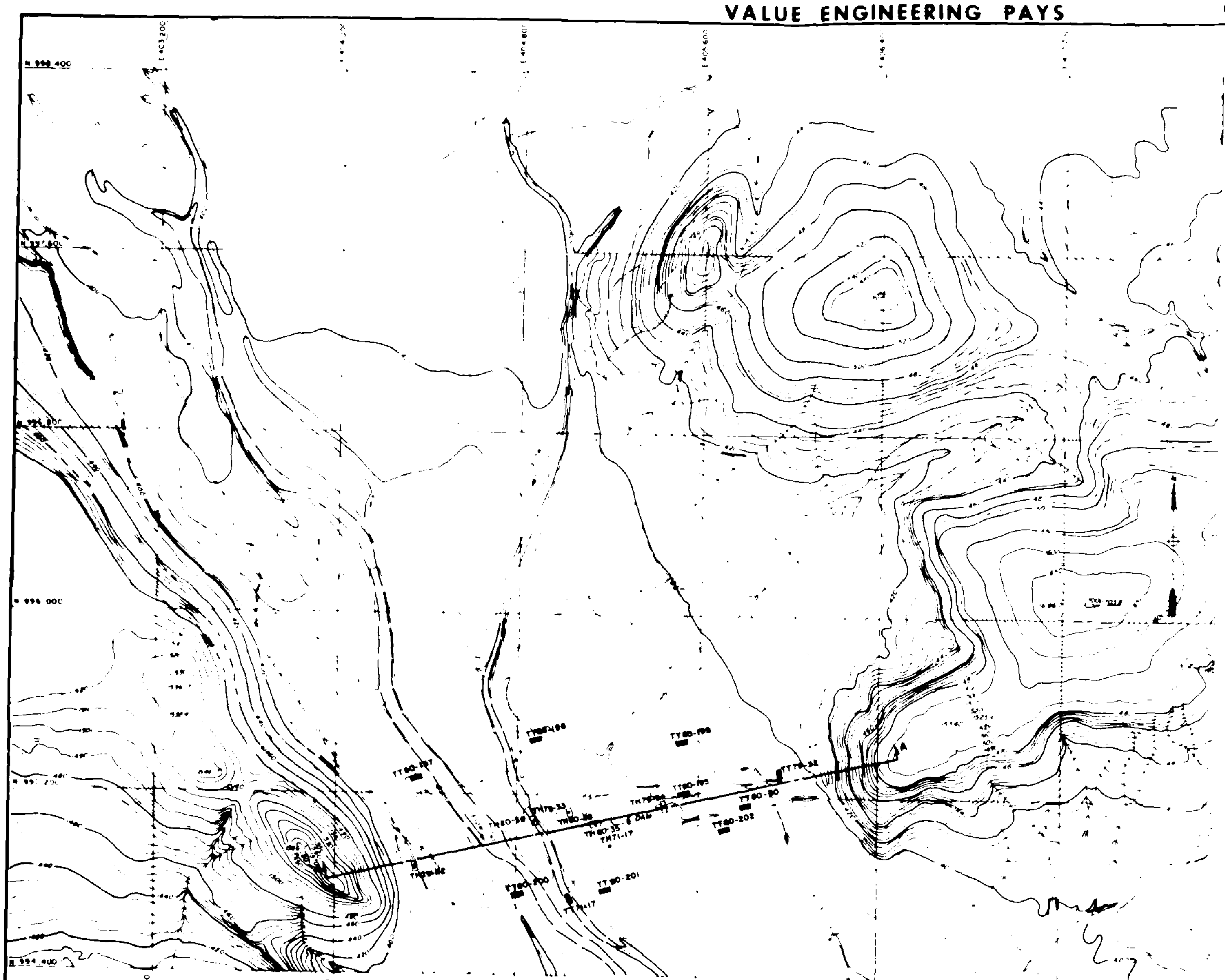
SAFETY PAYS

VALUE ENGINEERING PAYS



SAFETY PAYS

VALUE ENGINEERING PAYS



TEST TRENCHES

NO	DEPTH BELOW GROUND SURFACE	DATE OF EXCAVATION	REMARKS
71-17	10	3-71	TRENCHED TO 10 FEET BY DOZER
80-30	15	1-80	TRENCHED TO 4 FEET BY DOZER
79-32	10	12-79	TRENCHED TO 5 FEET BY DOZER
80-195	23	9-80	TRENCHED TO 23 FEET BY DOZER
80-197	12	9-80	TRENCHED TO 12 FEET BY DOZER
80-198	23	9-80	TRENCHED TO 19 FEET BY DOZER
80-199	19	9-80	TRENCHED TO 18 FEET BY DOZER
80-200	15	9-80	TRENCHED TO 13 FEET BY DOZER
80-201	33	9-80	TRENCHED TO 18 FEET BY DOZER
80-202	27	9-80	TRENCHED TO 15 FEET BY DOZER

TEST HOLES

NO	DEPTH BELOW GROUND SURFACE	DATE OF EXCAVATION AND DRILLING	REMARKS
71 17	10	2-71	[illegible]
79-32	26	12-79	TRENCHED TO 13 FEET, THEN DRILLED
79-33	34	12-79	TRENCHED TO 9 FEET, THEN DRILLED
79-34	24	12-79	TRENCHED TO 18 FEET, THEN DRILLED
80-35	97	1-80	TRENCHED TO 18 FEET, THEN DRILLED
80-36	82	8-80	TRENCHED TO 13 FEET, THEN DRILLED
80-38	77	8-80	TRENCHED TO 20 FEET, THEN DRILLED

PLAN OF EXPLORATION

100 0 100 200 300 400 500

CONTOUR INTERVAL 4 FEET

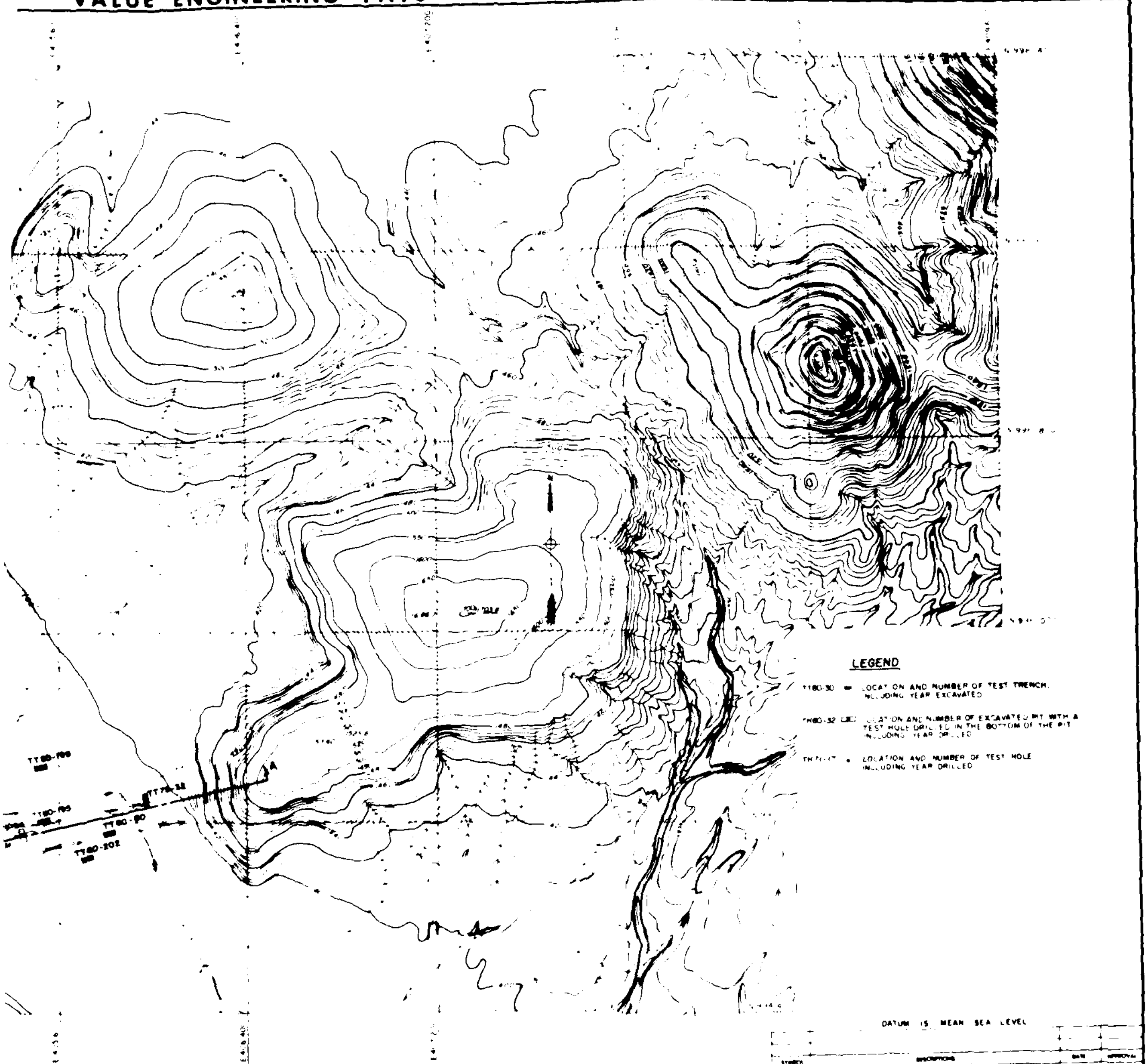
SCALE 1 IN = 200 FT

NOTES:

1. TEST HOLES WERE DRILLED BY A BUCKET TYPE POWER [illegible] DIAMETER WAS 24 INCHES
2. TEST TRENCHES WERE EXCAVATED WITH A BACKHOE AN[illegible]
3. FOR LOGS OF TEST TRENCHES AND HOLES SEE [illegible]
4. FOR PROFILE A-A SEE SHEET 4

SAFETY PAYS

VALUE ENGINEERING PAYS



LEGEND

TT80-30 — LOCATION AND NUMBER OF TEST TRENCH, INCLUDING YEAR EXCAVATED

TH80-32 — LOCATION AND NUMBER OF EXCAVATED PIT WITH A TEST HOLE DRILLED IN THE BOTTOM OF THE PIT INCLUDING YEAR DRILLED

TH — LOCATION AND NUMBER OF TEST HOLE INCLUDING YEAR DRILLED

PLAN OF EXPLORATION

SCALE 1 IN. = 200 FT

NOTES:

1. TEST HOLES WERE DRILLED BY A BUCKET TYPE POWER AUGER. THE BUCKET DIAMETER WAS 24 INCHES
2. TEST TRENCHES WERE EXCAVATED WITH A BACKHOE AND/OR D-8 DOZER
3. FOR LOGS OF TEST TRENCHES AND HOLES SEE SHEETS [illegible]
4. FOR PROFILE A-A SEE SHEET [illegible]

DATUM IS MEAN SEA LEVEL

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

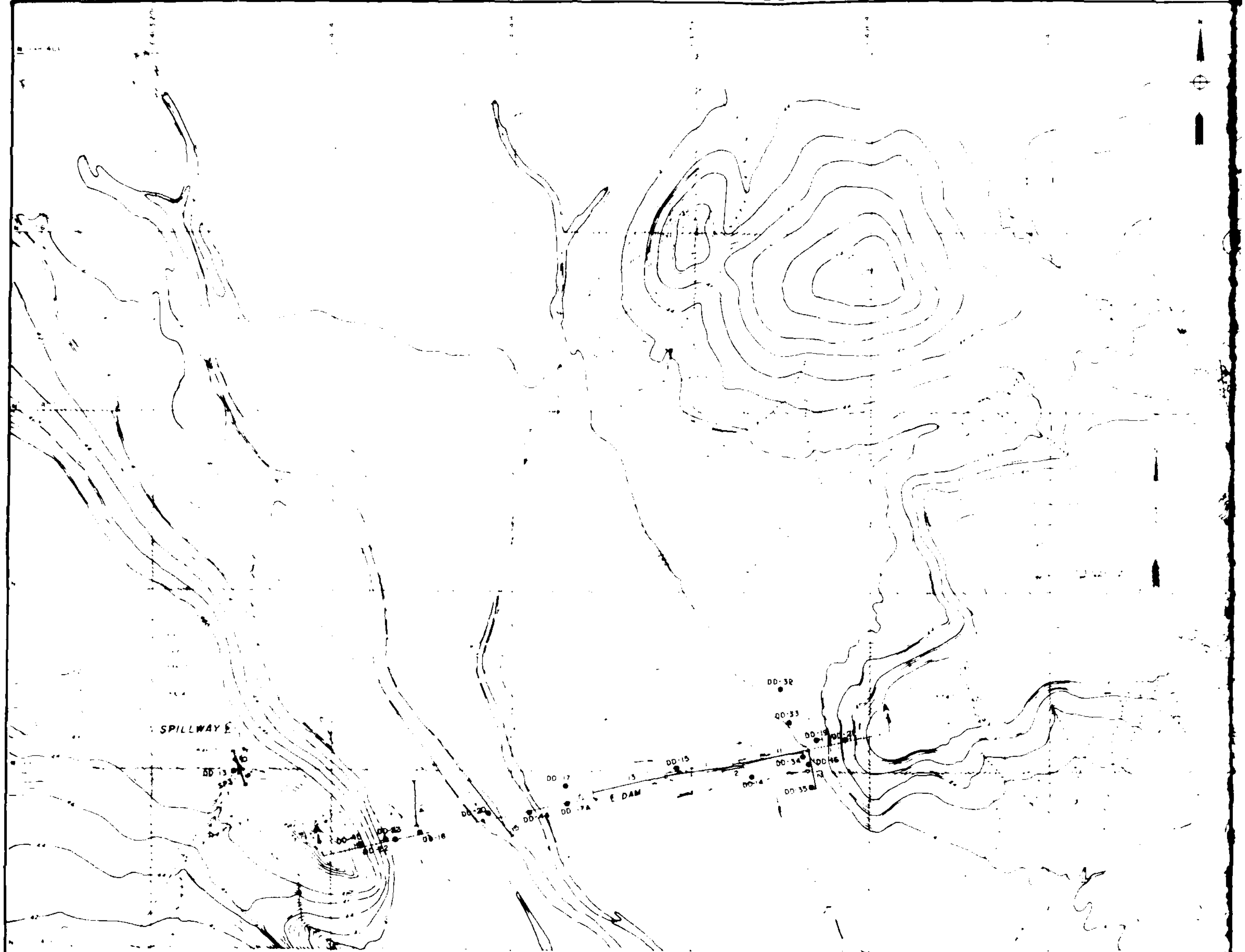
GILA RIVER BASIN

NEW RIVER DAM

DAM EMBANKMENT FOUNDATION
SOILS EXPLORATION PLAN

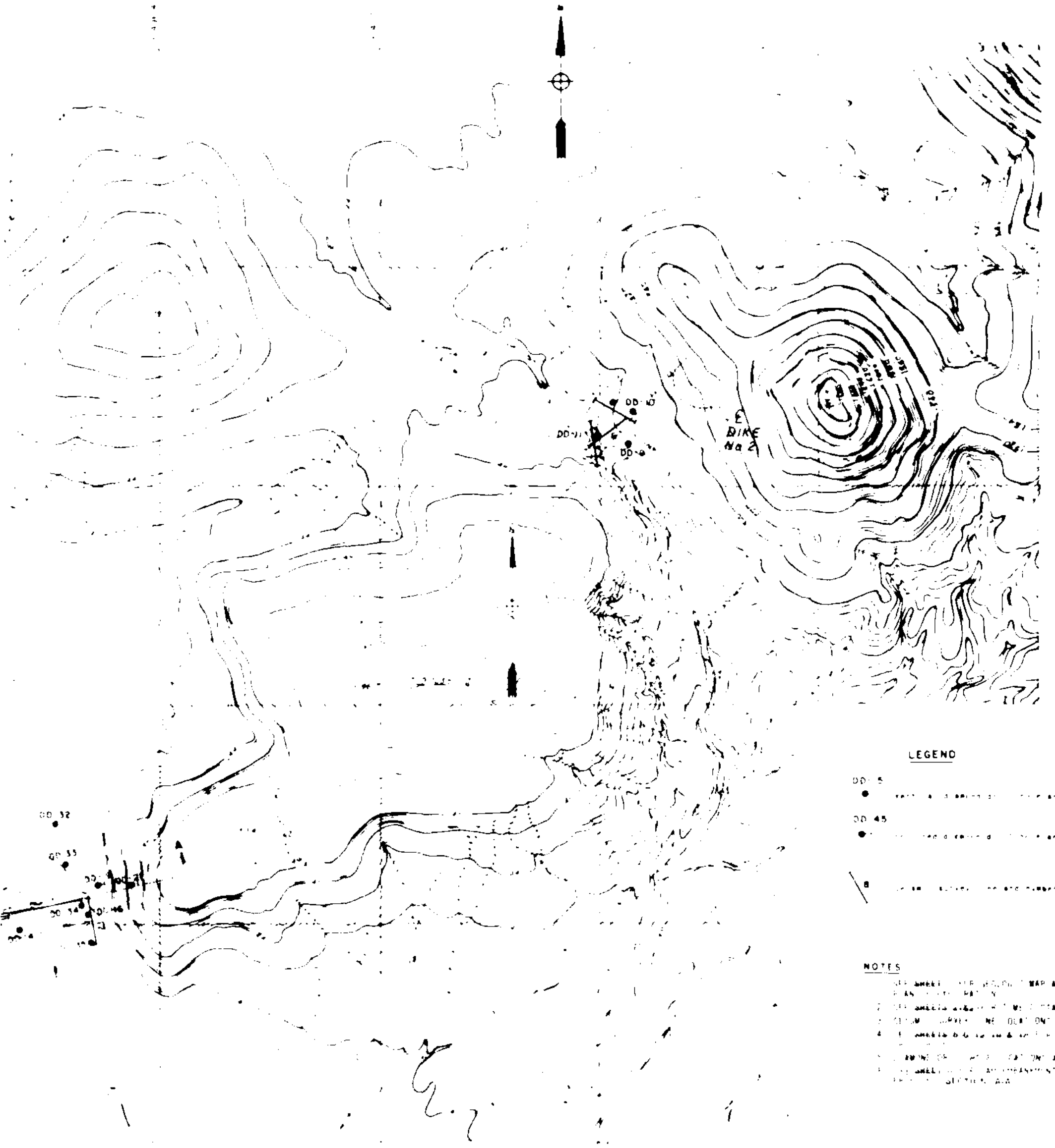
DESIGNED BY:
DRAWN BY: LBQ
CHECKED BY: ATR
SUBMITTED BY:
DATE APPROVED
SPEC. NO. DACW09-83-B-0016
DISTRICT FILE NO. [illegible]
SHEET 24

SAFETY PAYS



REFRACTIVE SEISMIC SURVEYS PLAN

LUE ENGINEERING PAYS



LEGEND

DD 5

DD 45

NOTES

DATUM IS MEAN SEA LEVEL

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

U S ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

A RIVER BASIN

ARIZONA

NEW RIVER DAM

REFRACTIVE SEISMIC SURVEYS

PLAN

DESIGNED BY

DRAWN BY
LBG

CHECKED BY

SUBMITTED BY

DATE
APPROVED

SPEC NO

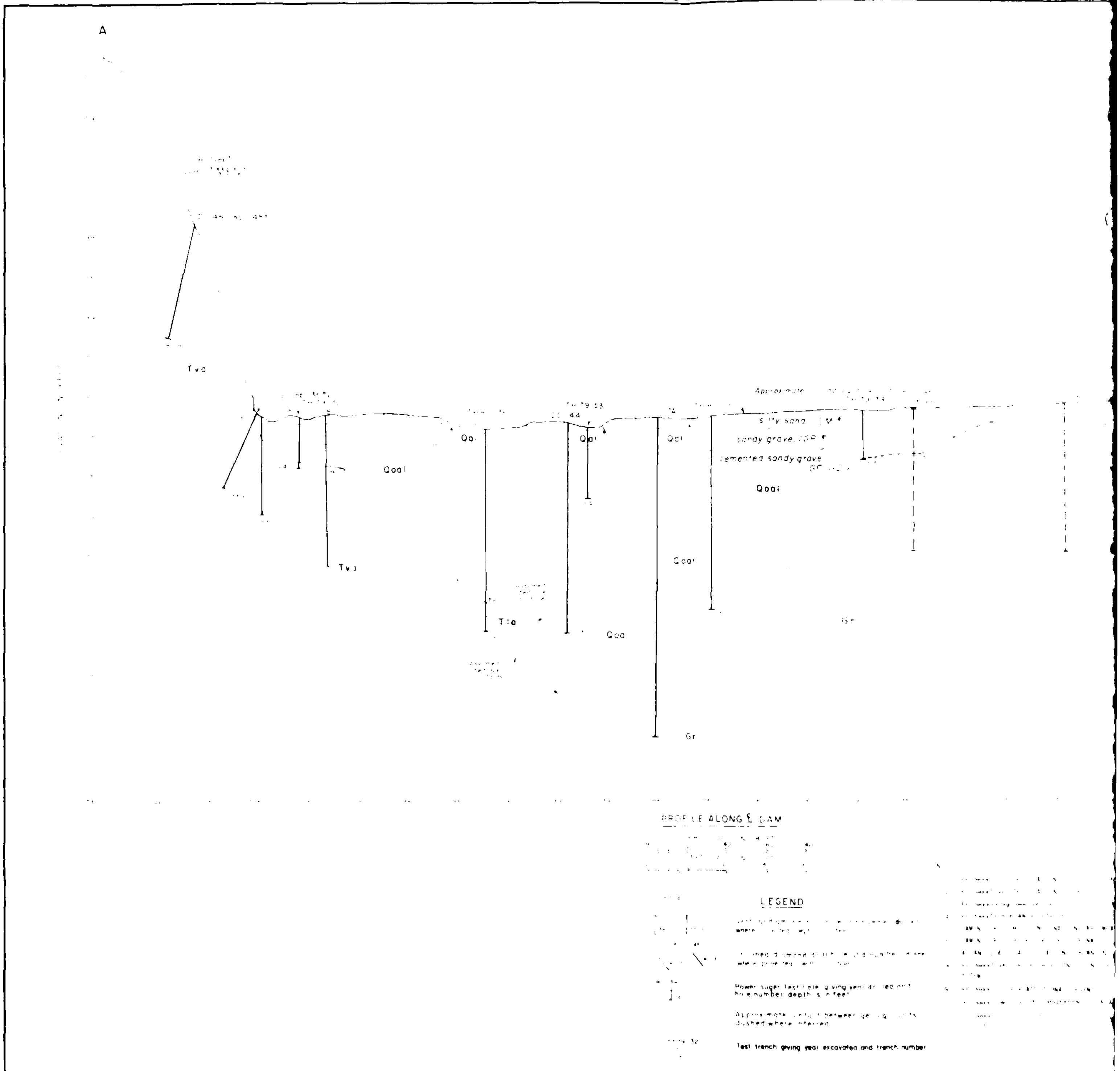
DISTRICT FILE NO

SHEET

SEISMIC SURVEYS PLAN

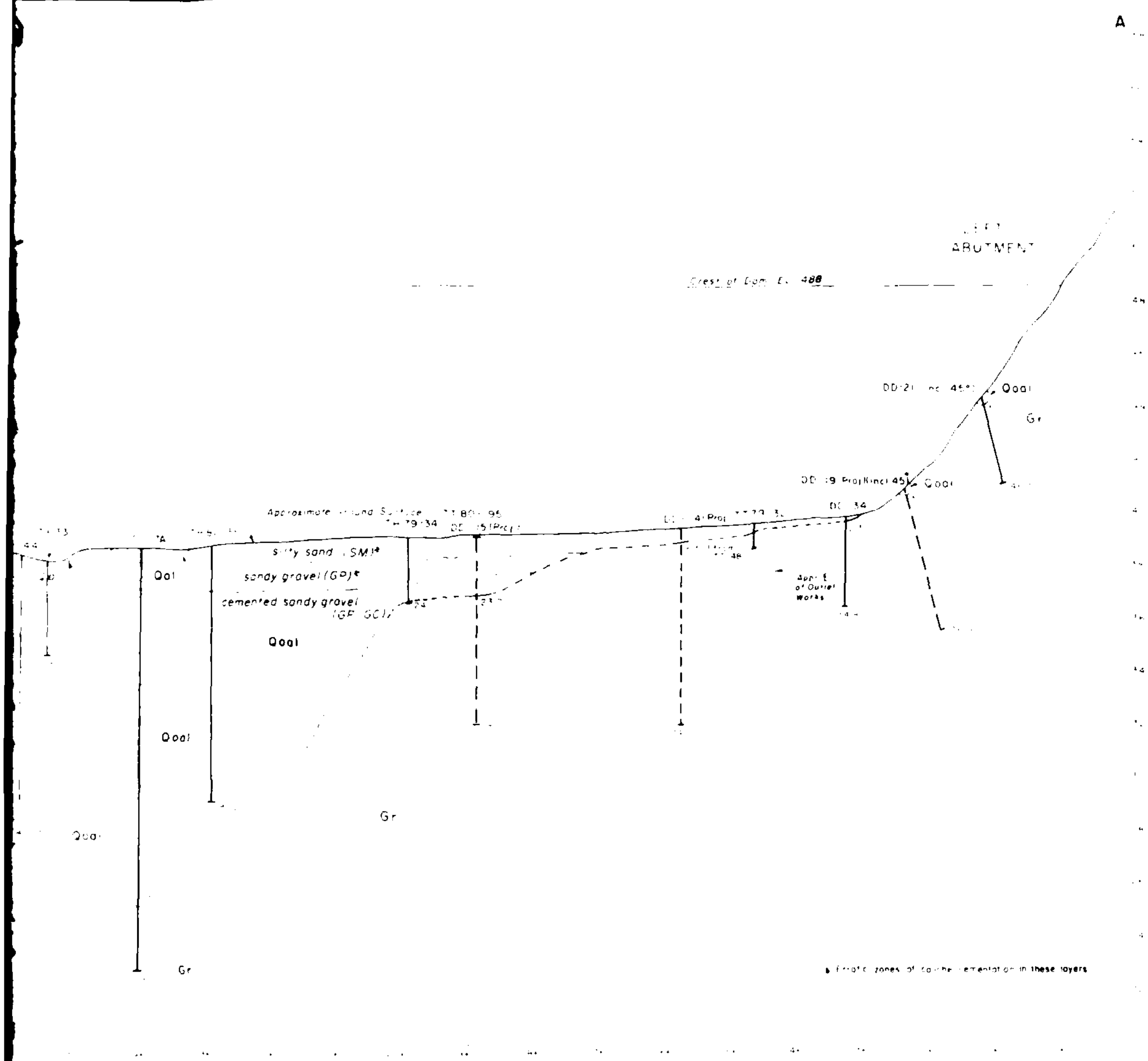
SAFETY PAYS

VALUE ENGINEERING PAYS



SAFETY PAYS

VALUE ENGINEERING PAYS



PROFILE ALONG ℄ DAM

LEGEND

Vertical diamond drill hole and number, dashed where projected. Depth is in feet

Inclined diamond drill hole and number, dashed where projected. Depth is in feet

Power auger test hole, giving year drilled and hole number, depth is in feet

Approximate contact between geologic units, dashed where inferred

TT 79-32 Test trench, giving year excavated and trench number

NOTES

1. SEE SHEET [illegible]
2. SEE SHEET [illegible]
3. SEE SHEETS [illegible]
4. SEE SHEETS [illegible]
5. DIAMOND DRILL HOLE INCLINATIONS ARE MEASURED FROM HORIZONTAL
6. DIAMOND DRILL HOLES [illegible]
7. [illegible]
8. SEE SHEET [illegible] SYSTEM
9. SEE SHEET [illegible] FOR ADDITIONAL LEGEND
10. SEE SHEET [illegible]

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

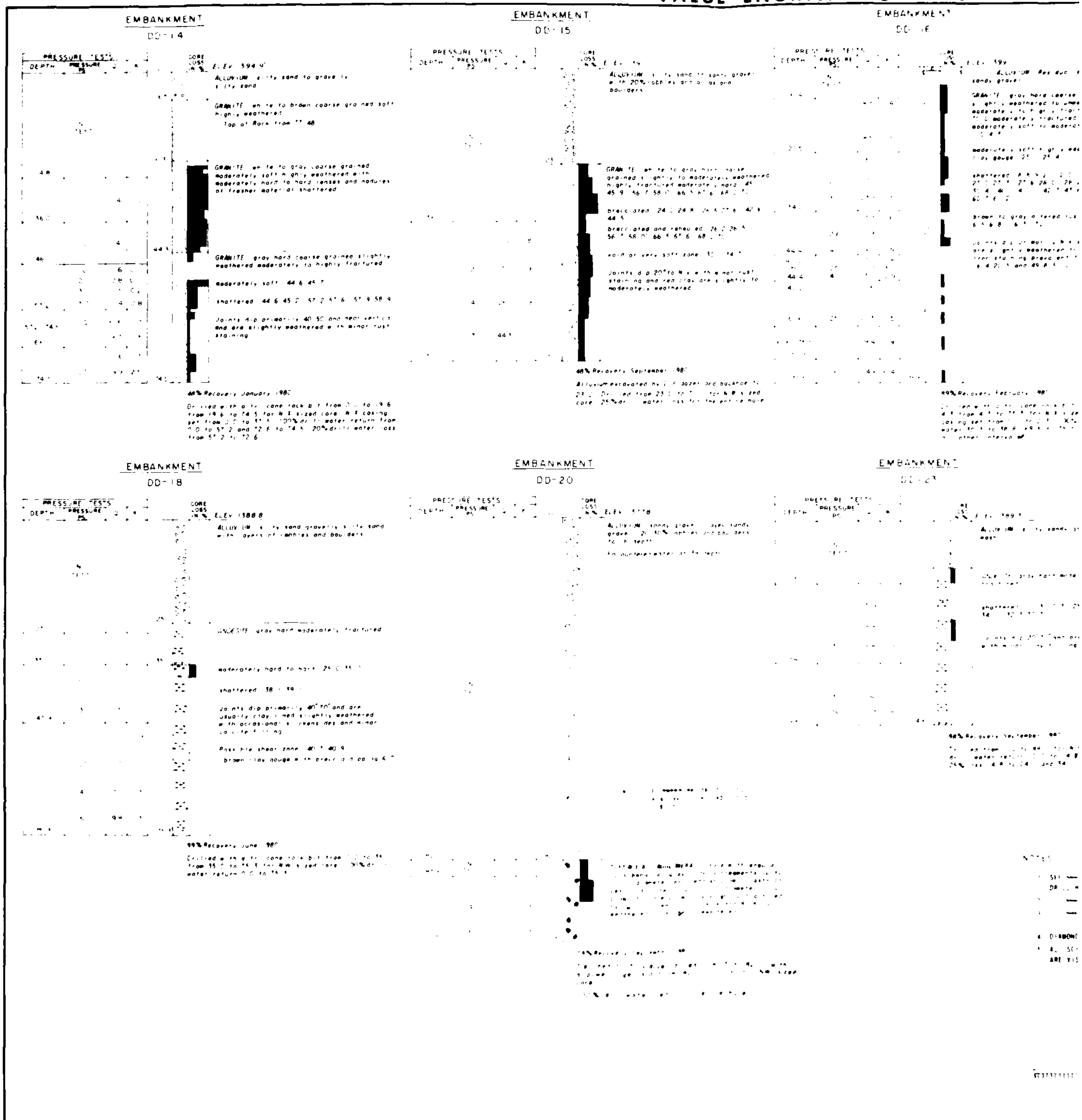
DESIGNED BY: RLT
DRAWN BY: G.T.
CHECKED BY: ATR

NEW RIVER DAM
DAM EMBANKMENT FOUNDATION
GEOLOGIC PROFILE

SUBMITTED BY
APPROVED
SPEC. NO. DACW09-
DISTRICT FILE NO. 241/229
SHEET 4 OF 80

SAFETY PAYS

VALUE ENGINEERING PAYS



SAFETY PAYS

ALUE ENGINEERING PAYS

EMBANKMENT

DD-16

PRESSURE TESTS

CORE LOSS IN % ELEV 1199

ALLUVIUM Residual soil silty sandy gravel

GRANITE gray hard coarse grained slightly weathered to unweathered moderately to highly fractured to 70.0 moderately fractured 70.0 to 75.5 moderately soft to moderately hard

moderately soft highly weathered with clay gouge 25 - 25.4

shattered 8.8 9.2 12.0 12.1 25.1 25.4 27.0 27.1 27.6 28.0 28.2 28.6 30 30.4 40 41 42 43.6 60.0 60 60 61.2

brown to gray altered rust stained 6.5 6.8 16.9

Joints dip primarily 30°60° and are slightly weathered to unweathered iron staining prevalent from 8.8 to 16.4 20.5 and 49.8 5.0

89% Recovery February 1980

Drilled with a tri cone rock bit from 0.0 to 4.5 from 4.5 to 75.5 for N X sized core. N X casing set from 0.0 to 2.5. 100% loss of drill water 38.5 to 38.8, 69.5 to 75.5. 50-100% return all other intervals

EMBANKMENT

DD-23

PRESSURE TESTS

CORE LOSS IN % ELEV 1189.1

ALLUVIUM silty sandy gravel and slope wash

ANDESITE gray hard moderately to highly fractured

shattered 12.1 29 33.0 33.7 34 32

Joints dip 20°70° and are slightly weathered with minor clay filling no rust staining

98% Recovery September 1980

Drilled from 0.0 to 49.1 for N X sized core. 100% drill water return 0.0 to 14.8 and 24 to 34. 25% loss 14.8 to 24.1 and 34 to 49.1

EMBANKMENT

DD-17a (17)

DEPTH	PRESSURE PSI	Q	K
NO TEST			
46			
5	NO TEST		
	21	.04	.12
	40	.08	.04
6			

CORE LOSS IN % ELEV 1386.5

ALLUVIUM

0.0 - 25' silty sand, sandy gravel with cobbles and boulders

25 - 36.0' clayey sandy gravel to gravelly sandy clay

encountered water at 70' depth in DD-17

GRANITE white to grey, moderately soft to moderately hard, moderately to highly weathered, highly fractured

shattered 146.1 148.0

Joints dip generally 30 60° mostly parallel with some intersecting 70° to near vertical, are moderately to highly weathered with red and green clay and some slickensides

estimate top of rock at 136 from drill action and cuttings

91% Recovery January 1981

DD-17A Drilled with a tri-cone rock bit from 0.0 146.1, from 146.1-161.1' for N X sized core. 100% drill water return 0.0 to 161.1

DD-17 Trenched to 20 by dozer, drilled from 20 to 77.0 with a power auger and from 77.0' to 106.0' with a tri-cone rock bit. No bedrock encountered

EMBANKMENT

DD-44

PRESSURE TESTS

DEPTH PRESSURE Q K

CORE LOSS IN % ELEV 1876

ALLUVIUM

0.0 16.0 silty sand gravelly silty sand with layers of cobbles and boulders

16.0 106.0 gravelly sandy clay to clayey sandy gravel

NO TEST

106.0

January 1981

Drilled with a tri cone rock bit and heavy revert mud mix from 0.0 to 106.0. No bedrock encountered

LEGEND

WATER PRESSURE TEST

DEPTH

PRESSURE pounds per square inch (PSI)

Q

K

NO TEST interval not tested

GRAVITY Packer not used during testing

CORE LOSS The % of core loss for each interval drilled in bedrock is shown by the bar graph. A completely shaded area represents 100% core loss and an unshaded area represents 0% core loss.

Stratigraphic symbol to left of core loss column represents the geologic unit described on the log.

CLASSIFICATION OF FRACTURES

Shattered fracture spacing of less.
fracture spacing.
to 12 inch fracture spacing.
12 inch fracture spacing.

HARDNESS DEFINITIONS

very friable.
can be
can be broken with light hammer
can be broken with moderate to heavy hammer

NOTES

1. SEE FOR THE GEOLOGIC MAP AND DIAMOND DRILL HOLE PLAN OF EXPLORATION
2.
3.
4. DIAMOND DRILL HOLE ELEVATIONS ARE SURVEYED
5. ALL SOIL CLASSIFICATIONS SHOWN ON THIS SHEET ARE VISUAL

DATUM IS MEAN SEA LEVEL

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

U S ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY: P.T.

DRAWN BY: M.P.

CHECKED BY:

GILA RIVER BASIN
ARIZONA AND (INCLUDING NEW RIVER)
NEW RIVER DAM
DAM EMBANKMENT
GEOLOGIC LOGS OF DIAMOND
DRILL HOLES

SUBMITTED BY

DATE APPROVED

SPEC NO DACW09-83-B-0016

DISTRICT FILE NO 241/230

SHEET 5 OF 80

SAFETY PAYS

TH80-35

SILTY SAND [illegible]

SANDY GRAVEL: brown, dry, dense, 2[illegible] cobbles [illegible] boulders [illegible]

GRAVEL/CLAYEY GRAVEL: reddish brown, moist, dense, [illegible] cobbles [illegible]

SANDY GRAVEL/CLAYEY SANDY GRAVEL

SANDY GRAVEL/SILTY SANDY GRAVEL

CLAYEY SANDY GRAVEL

SANDY GRAVEL/CLAYEY SANDY GRAVEL

CLAYEY SANDY GRAVEL

SANDY GRAVEL/CLAYEY SANDY GRAVEL

CLAYEY SANDY GRAVEL: dark brown [illegible]

SANDY GRAVEL/CLAYEY SANDY GRAVEL: reddish brown, cemented, 5-[illegible] cobbles [illegible]

CLAYEY SANDY GRAVEL: dark brown, cemented, 5-[illegible] cobbles

SANDY GRAVEL/CLAYEY SANDY GRAVEL: dark brown

SILTY SANDY GRAVEL: dark brown, 5-10% cobbles

SANDY GRAVEL/SILTY SANDY GRAVEL: reddish brown, few cobbles to 9"

SILTY GRAVELLY SAND: reddish brown

CORE BARREL, FLIGHT AUGER, BUCKET AUGER and GAC USED due to a nest of boulders

CLAYEY GRAVELLY SAND: reddish brown, 5-8% cobbles to 7"

SANDY GRAVEL/SILTY SANDY GRAVEL: reddish brown

SANDY GRAVEL/CLAYEY SANDY GRAVEL: reddish brown, cemented

SILTY SANDY GRAVEL: reddish brown, 9" cobbles

GRAVELLY SILTY SAND: dark brown, wet, 5-6% cobbles

SILTY SANDY GRAVEL: dark brown

DRILLING stopped due to nest of boulders

TH80-36

SANDY GRAVEL [illegible]

SANDY GRAVEL [illegible]

SANDY GRAVEL/CLAYEY SANDY GRAVEL [illegible]

GRAVELLY CLAYEY SAND [illegible]

SILTY GRAVELLY SAND

SANDY GRAVEL/SILTY SANDY GRAVEL

GRAVELLY SAND/SILTY GRAVELLY SAND

GRAVELLY SAND/SILTY GRAVELLY SAND: brown [illegible]

wet

SANDY GRAVEL: brown, auger speed alternates between fast and slow [illegible] through alternating soft and hard soil layers

GRAVELLY SAND/SILTY GRAVELLY SAND

SANDY GRAVEL/SILTY SANDY GRAVEL

GRAVELLY SAND/SILTY GRAVELLY SAND

CLAYEY SAND: wet [illegible]

GRAVELLY SAND/SILTY GRAVELLY SAND

SILTY GRAVELLY SAND

SANDY GRAVEL/SILTY SANDY GRAVEL

GRAVELLY SILTY SAND

SANDY GRAVEL/SILTY SANDY GRAVEL

GRAVELLY SILTY SAND

REMARKS: [illegible] change in auger speed implies drilling through alternating soft and hard soil layers

TH80-38

[illegible]

COARSE GRAINED SOILS

FINE GRAINED SOILS

NOT

GW

VERT SCALE

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE GRAINED SOILS More than half of material is larger than no. 200 sieve size	GRAVELS More than half of coarse fraction is larger than no. 4 sieve size	Clean gravels	GW	Well graded gravels, gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines
		Gravels with fines	GM	Silty gravels, gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
	SANDS More than half of coarse fraction is smaller than no. 4 sieve size	Clean sands	SW	Well graded sands, gravelly sands, little or no fines
			SP	Poorly graded sands, gravelly sands, little or no fines
		Sands with fines	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
FINE GRAINED SOILS More than half of material is smaller than no. 200 sieve size	SILTS AND CLAYS	Low liquid limit	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	Organic silts and organic silty clays of low plasticity
		High liquid limit	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity, organic silts
Highly organic soils			Pt	Peat and other highly organic soils

NOTES

1. Boundary Classification: Soils possessing characteristics of two groups are designated by combinations of group symbols. For example, GW-GC well graded gravel-sand mixture with clay binder.
2. All sieve sizes on this chart are U. S. Standard.
3. The terms "silt" and "clay" are used respectively to distinguish materials exhibiting lower plasticity from those with higher plasticity. The minus no. 200 sieve material is silt if the liquid limit and plasticity index plot below the "A" line on the plasticity chart (Table VI, Military Standard 619B) and is clay if the liquid limit and plasticity index plot above the "A" line on the chart.
4. For a complete description of the Unified Soil Classification System see "Military Standard 619B" dated 20 March 1976.

LEGEND

TT-BC-39	LOCATION AND NUMBER OF TEST TRENCH
TH-BC-35	LOCATION AND NUMBER OF TEST HOLE
MC	FIELD MOISTURE CONTENT IN PERCENT OF DRY WEIGHT
LL	LIQUID LIMIT
PI	PLASTICITY INDEX (LIQUID LIMIT MINUS PLASTIC LIMIT)
NP	NONPLASTIC
4	PERCENT OF MATERIAL BY WEIGHT PASSING NO. 4 SIEVE
200	PERCENT OF MATERIAL BY WEIGHT PASSING NO. 200 SIEVE
N	NUMBER OF BLOWS OF A 140 POUND DROPHAMMER FALLING 30 INCHES REQUIRED TO DRIVE A SAMPLING SPOON ONE FOOT. OUTSIDE DIAMETER OF SPOON IS 2 INCHES, INSIDE DIAMETER IS 1 3/8 INCHES. PROCEDURE IS CALLED STANDARD PENETRATION TEST
w	DEPTH TO WATER
Ex	MATERIALS TO BE EXCAVATED FROM THE FOUNDATION
C	BLENDED MATERIAL SUITABLE FOR CORE
PS	BLENDED MATERIAL SUITABLE FOR PERVIOUS SHELL
NS	NO SAMPLE
*	VISUAL CLASSIFICATION
+	MINUS 3 INCH GRADATION SAMPLE
++	MINUS 6 INCH GRADATION SAMPLE

NOTES

1. SEE SHEET 24 FOR LOCATION OF TEST HOLES AND TRENCHES.
2. PERCENTAGE OF COBBLES, BOULDERS AND MAXIMUM SIZES WHERE INDICATED WERE VISUALLY ESTIMATED.
3. ALL GRADATIONS EXCEPT THOSE INDICATED WERE PERFORMED ON MINUS 3 INCH SAMPLES.
4. DATES OF DRILLING: JANUARY 1977–APRIL 1977, DECEMBER 1979–FEBRUARY 1980, AND AUGUST 1980–SEPTEMBER 1980.

DATUM: MEAN SEA LEVEL

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

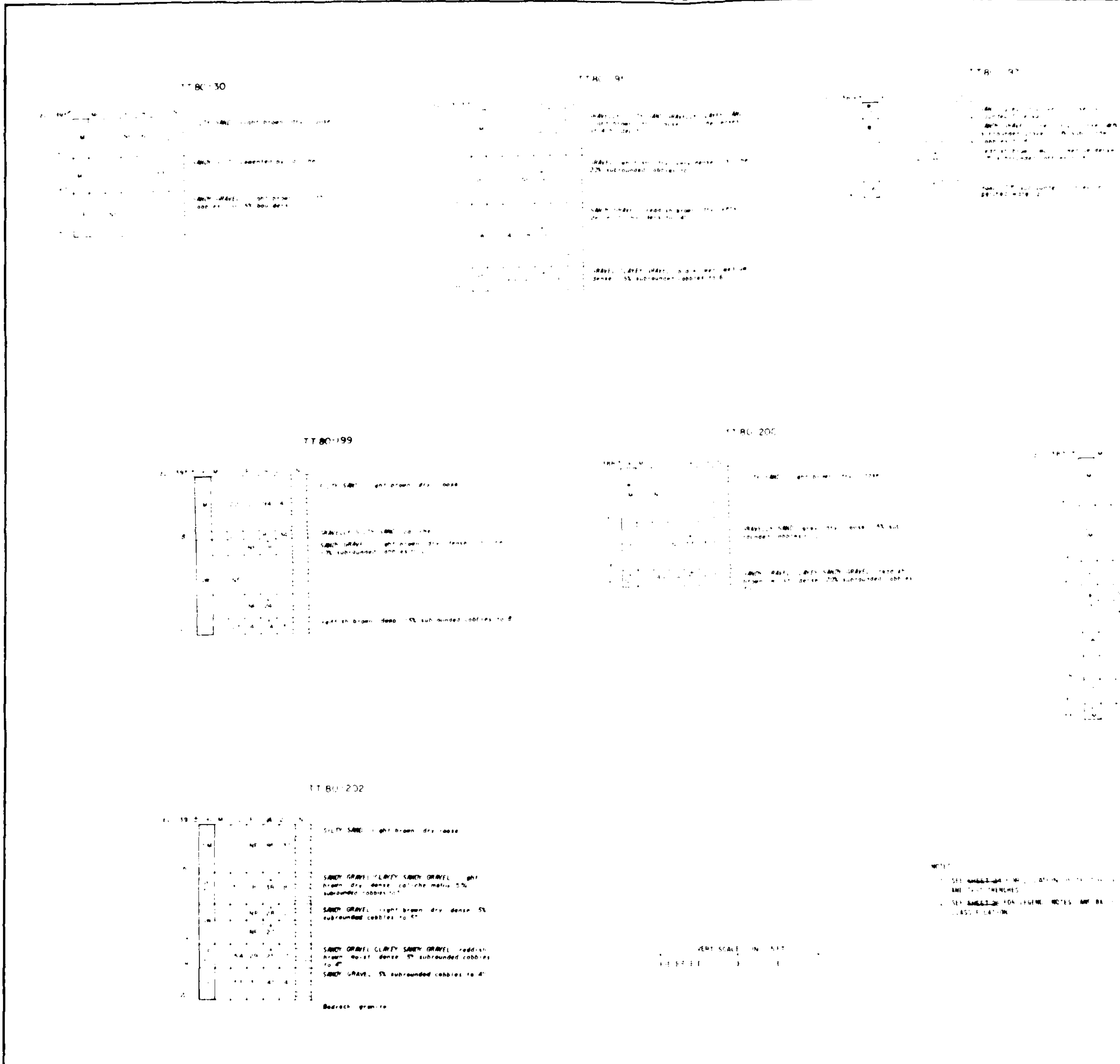
REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

GILA RIVER BASIN
PHOENIX, ARIZONA AND VICINITY (INCLUDING NEW RIVER)

NEW RIVER DAM
DAM EMBANKMENT FOUNDATION
SOIL LOGS

DESIGNED BY:
DRAWN BY:
CHECKED BY: ATR
SUBMITTED BY:
DATE APPROVED:
SPEC. NO. DACW09-83-B-0014
DISTRICT FILE NO. 74/25
SHEET 28 OF 80


TT 80-30
TT 80-199
TT 80-202
SILTY SAND light brown dry loose
SANDY GRAVEL light brown dry dense 5% subrounded cobbles to 5"
SANDY GRAVEL 5% subrounded cobbles to 4"
Bedrock granite
VERT SCALE IN FEET

VALUE ENGINEERING PAYS

TT80-197

SAND grey dry very loose sub rounded fine sand

SANDY GRAVEL grey dry loose 40% subrounded gravel 7% subrounded cobbles to 8"

reddish brown moist medium dense 20% subrounded cobbles to 8"

damp 20% subrounded cobbles to ..., perched water at

TT80-198

SILTY SAND light brown dry loose

SANDY GRAVEL grey dry dense 15% sub rounded cobbles to 5"

brown damp 40% gravel 15% subrounded cobbles to 6"

reddish brown, dense

SAND CLAYEY SAND reddish brown damp, dense

SANDY GRAVEL/CLAYEY SANDY GRAVEL reddish brown damp dense 5% subrounded cobbles to 4" perched water at 22

TT80-201

SILTY SAND tan dry loose

SANDY GRAVEL SILTY SANDY GRAVEL light brown dry dense 0% cobbles to 8"

SANDY GRAVEL

SANDY GRAVEL CLAYEY SANDY GRAVEL reddish brown, moist dense 30% subrounded cobbles to 6"

CLAYEY SAND reddish brown damp dense

SANDY GRAVEL CLAYEY SANDY GRAVEL reddish brown damp dense 7% subrounded cobbles to 8"

SANDY GRAVEL SILTY SANDY GRAVEL

NOTES

1. SEE ~~SHEET 24~~ FOR LOCATION OF TEST PITS AND TEST TRENCHES
2. SEE ~~SHEET 26~~ FOR LEGEND, NOTES AND BASIS OF CLASSIFICATION

DATUM IS MEAN SEA LEVEL

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

U S ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY

DRAWN BY
J F B

CHECKED BY
ATR

GILA RIVER BASIN
PHOENIX ARIZONA AND VICINITY INCLUDING NEW RIVER

NEW RIVER DAM
DAM EMBANKMENT FOUNDATION
SOIL LOGS

SUBMITTED BY
DATE
APPROVED

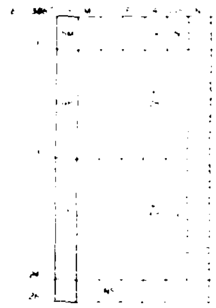
SPEC NO DACW09 83 B 0016

DISTRICT FILE NO

SHEET 27 OF 80

SAFETY PAYS

TH 79-32



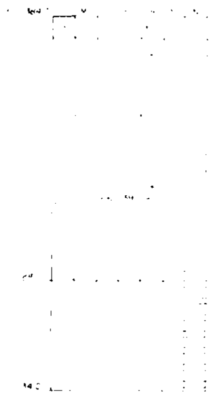
SILTY SAND light brown, dry, loose

SANDY GRAVEL light brown, dry, loose

SANDY GRAVEL, CLAYEY SANDY GRAVEL

BEDROCK

TH 79-33



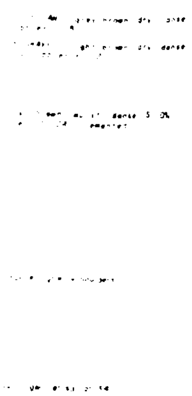
TH 79-34

CLAYEY SAND dark brown, cemented

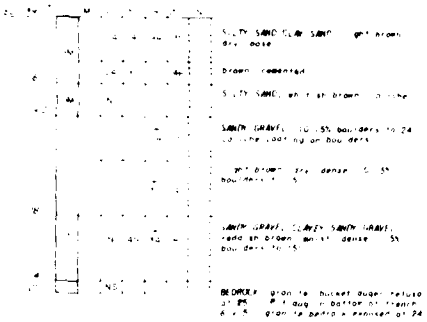
BEDROCK

Backhoe refusal

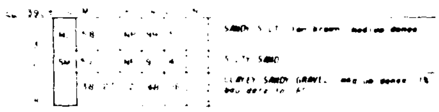
SAFETY PAYS

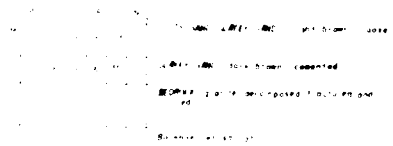


TH 79-34

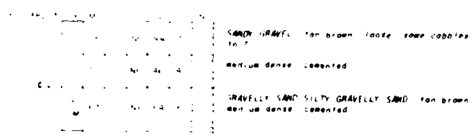


TH 71-17





TT 71-17



NOTES

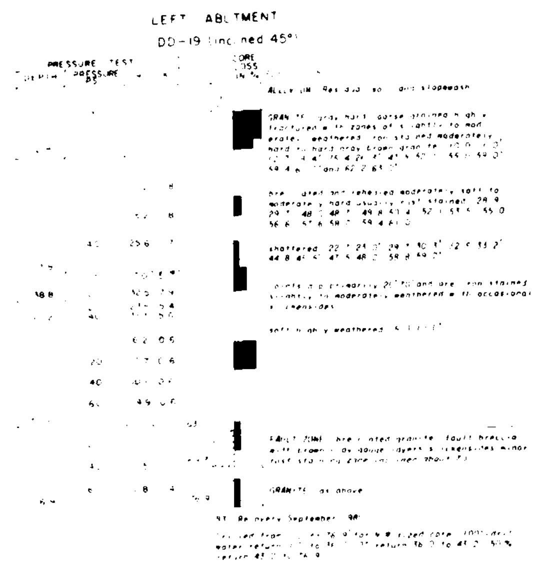
REVISIONS

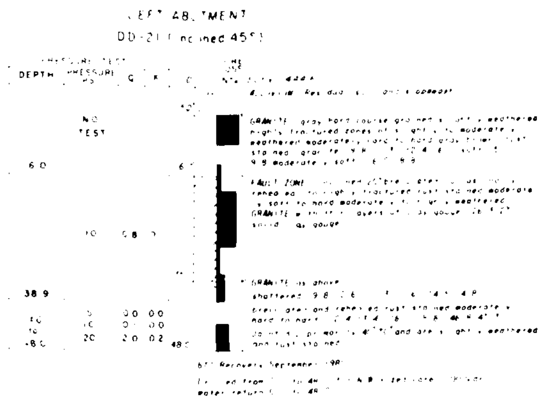
SYMBOL	DESCRIPTIONS	DATE	APPROVAL

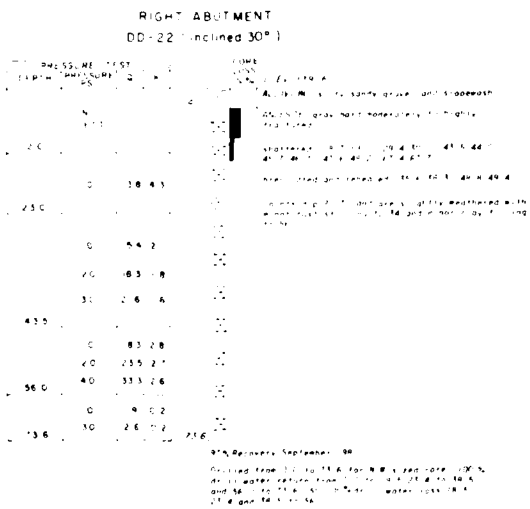
U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

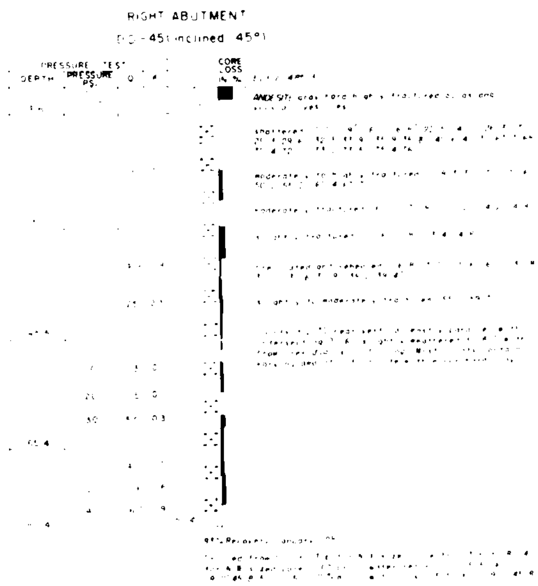
NEW RIVER DAM
DAM EMBANKMENT FOUNDATION
SOIL LOGS

DESIGNED BY
DRAWN BY
CHECKED BY ATR
SUBMITTED BY
DATE APPROVED
SPEC NO. DACW09-83-B-0016
DISTRICT FILE NO. 241-253
SHEET 38 OF 81


LEFT ABUTMENT
DD-19 (inclined 45°)
PRESSURE TEST
DEPTH
PRESSURE
CORE
LOSS
IN %
GRANITE as above
Recovery September


LEFT ABUTMENT
DD-21 (inclined 45°)
DEPTH
PRESSURE
NO
TEST
GRANITE
FAULT ZONE
GRANITE as above
shattered
Recovery September


RIGHT ABUTMENT
DD-22 (inclined 30°)
PRESSURE TEST
DEPTH
PRESSURE
CORE
LOSS
Recovery September


RIGHT ABUTMENT
DD-45 (inclined 45°)
PRESSURE TEST
DEPTH
PRESSURE
PSI
CORE
LOSS
IN %
ANDESITE

VALUE ENGINEERING PAYS

NOTES

1. SEE ... FOR THE GEOLOGIC MAP AND DIAMOND DRILL HOLE PLAN OF EXPLORATION
2. SEE ...
3. SEE ...
4. DIAMOND DRILL HOLE ELEVATIONS ARE SURVEYED
5. ALL SOIL CLASSIFICATIONS SHOWN ON THIS SHEET ARE VISUAL
6. DIAMOND DRILL HOLE INCLINATIONS ARE MEASURED FROM HORIZONTAL

DATUM IS MEAN SEA LEVEL

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

U S ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY: RLT

DRAWN BY:

CHECKED BY:

... RIVER BASIN

NEW RIVER DAM
DAM EMBANKMENT
(RIGHT AND LEFT ABUTMENTS)
GEOLOGIC LOGS OF DIAMOND
DRILL HOLES

SUBMITTED BY

DATE APPROVED

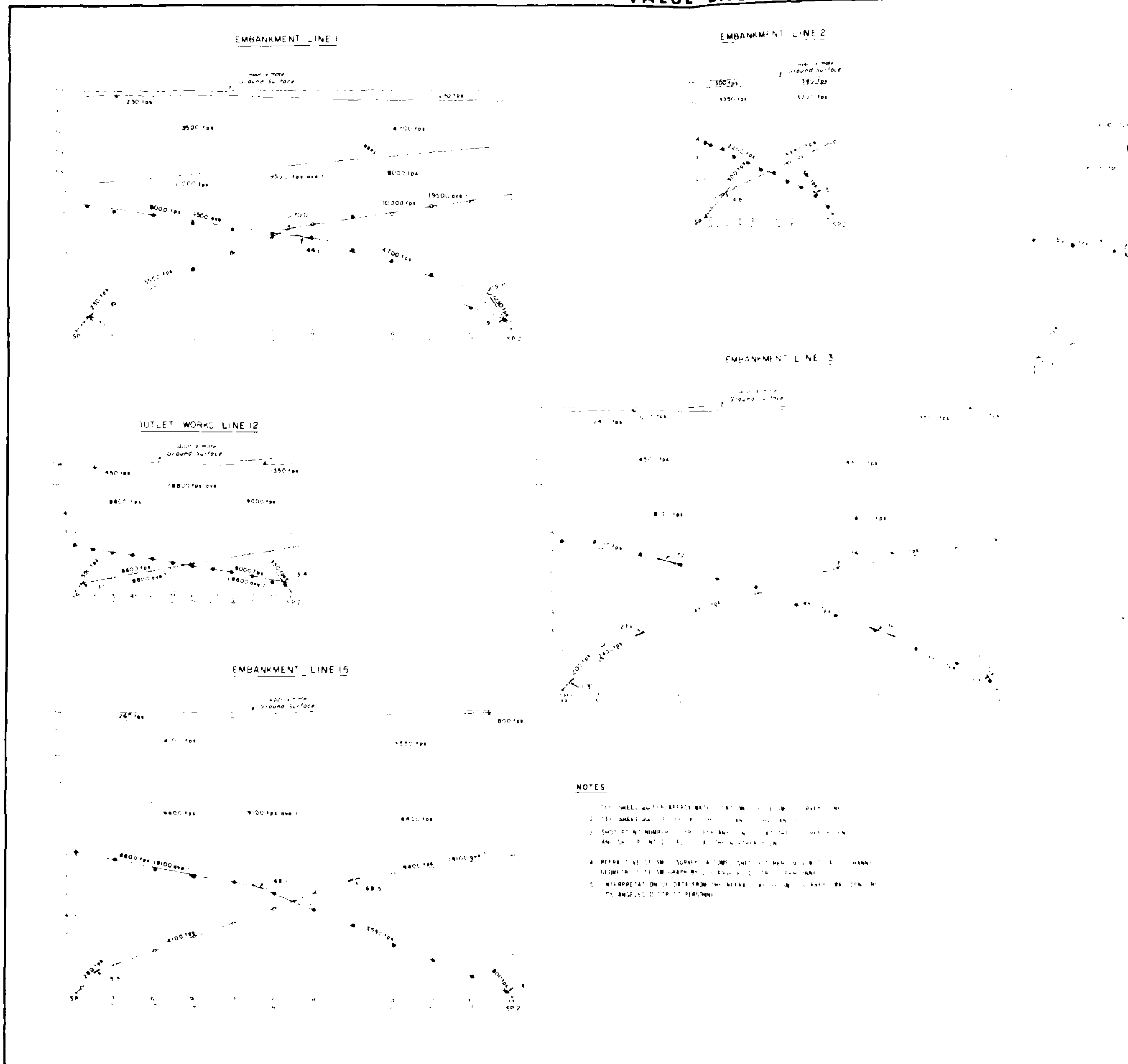
SPEC NO

DISTRICT FILE NO 241/23

SHEET 6 OF 80

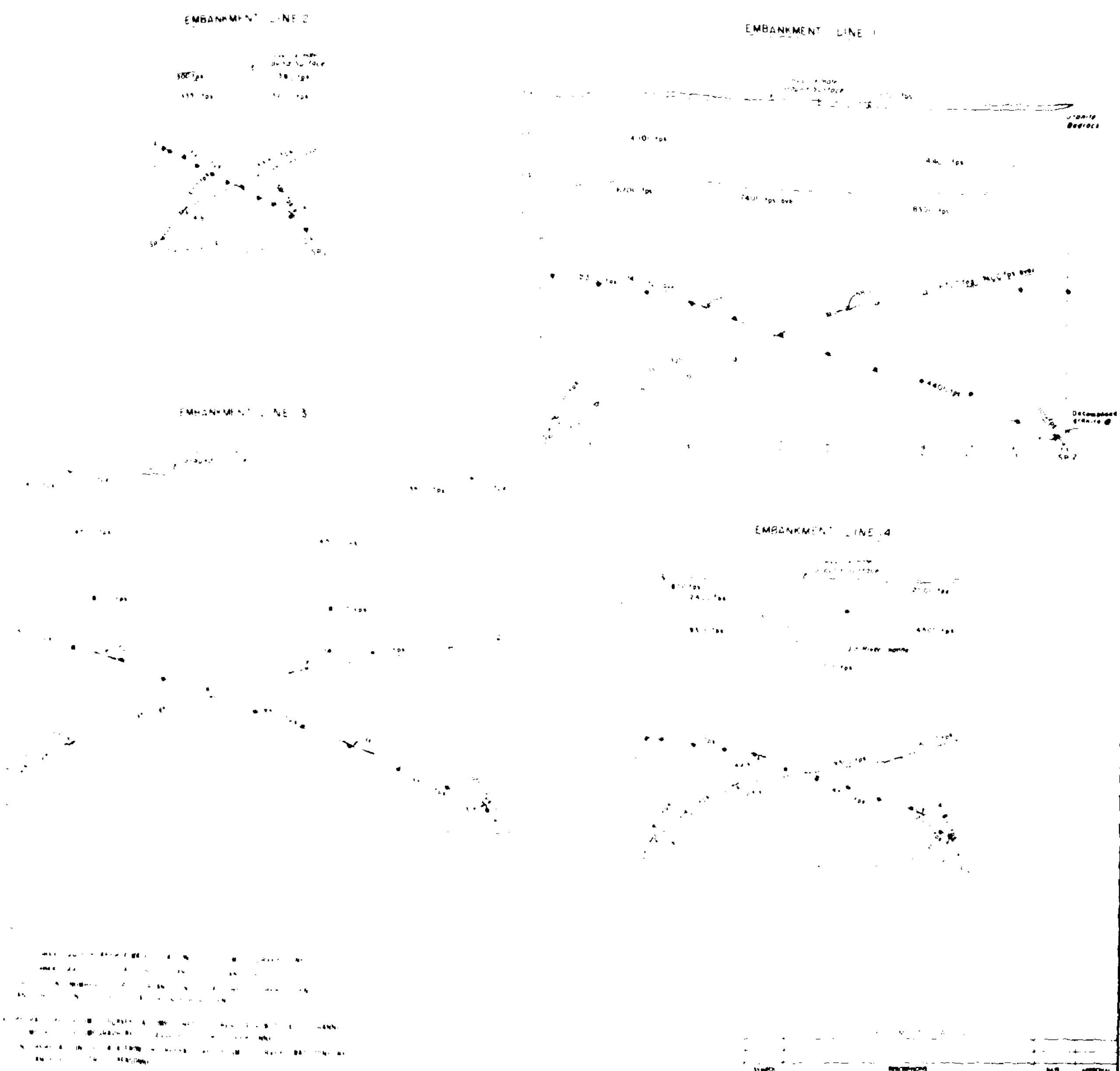
SAFETY PAYS

VALUE ENGINEERING PAYS



SAFETY PAYS

VALUE ENGINEERING PAYS

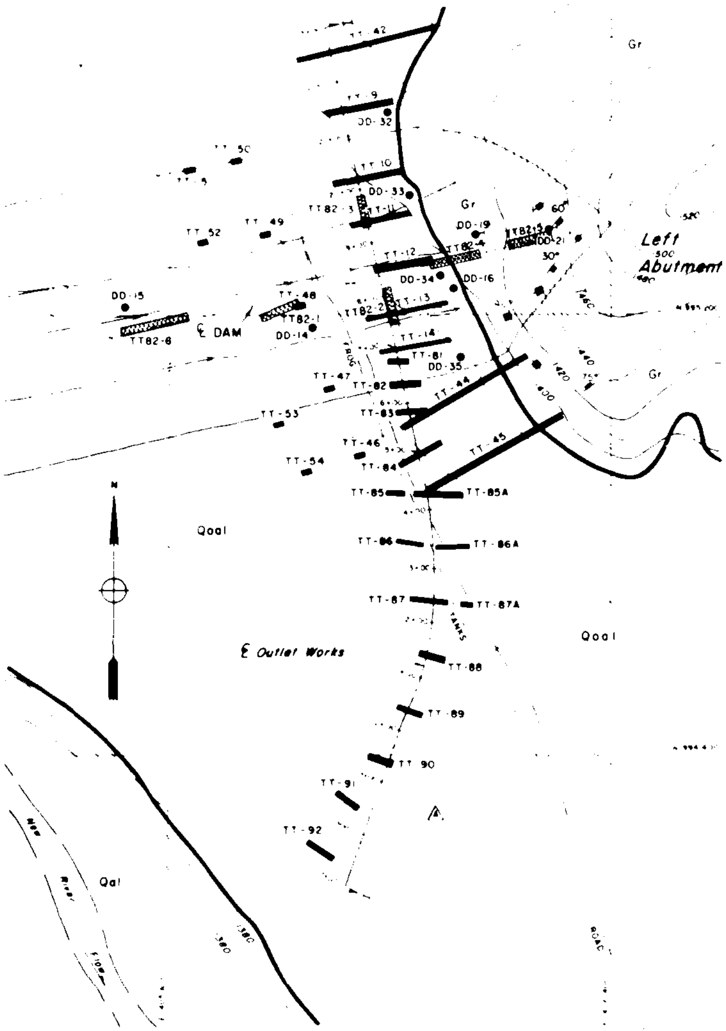


REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

NEW RIVER DAM
DAM EMBANKMENT AND OUTLET WORKS
REFRACTIVE SEISMIC SURVEYS
TIME DISTANCE CURVES AND PROFILES

SAFETY PAYS

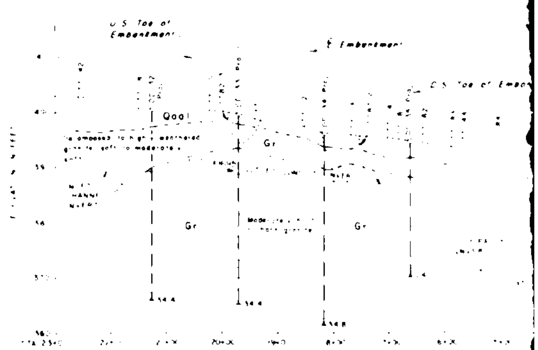


LEFT ABUTMENT, EMBANKMENT-TEST TRENCH PLAN

OUTLET WORKS-PLAN OF EXPLORATION

LOGS OF EMBANKMENT

TEST TRENCH NO	DEPTH TO BEDROCK	TOTAL DEPTH	DIMENSIONS	BEARING	
46		8	8 LONG 2 WIDE	N 61 E	0.0-6.0 SILTY ... 5 TO 2 IN DIAME...
47	6	6.5	12 LONG 2 WIDE	N 15 E	0.0-6.0 SILTY S... REFUSAL IN GRANITE...
48	6.5		16 LONG 2 WIDE	N 14 E	0.0-6.5 SILTY S... REFUSAL IN GRANITE...
49	5.5	6	21 LONG 2 WIDE	N 86 E	0.0-5.5 SANDY S... REFUSAL IN GRANITE...
50	10		22 LONG 2 WIDE	N 12 E	0.0-10.0 SANDY S... AT 10.0
51	—	8.5	25 LONG 2 WIDE	N 84 E	0.0-1.0 SILTY ... 6" IN DIAMETER
52		10.0	25 LONG 2 WIDE	N 18 E	0.0-6.0 SANDY S... COMPACTED REFUSAL...
53	—	8	25 LONG 2 WIDE	N 15 E	0.0-1.0 SANDY ... REFUSAL IN CALICHE...
54	—	9	25 LONG 2 WIDE	N 61 E	0.0-5.0 SANDY ... DIAMETER ...



PROFILE ALONG ℄ OU...

NOTES

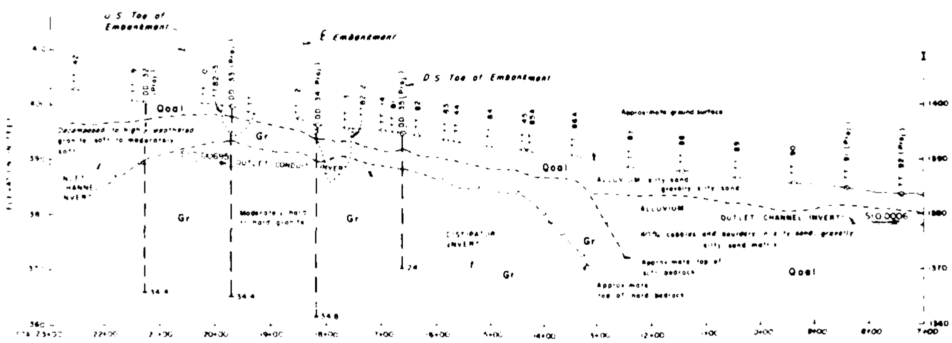
1. SEE SHEET ... FOR THE GEOLOGIC MAP AND DIAMOND ... PLAN OF EXPLORATION
2. SEE SHEET ... FOR LOGS OF TEST TRENCHES 9 THROUG...
3. SEE SHEETS ... FOR LOGS OF TEST TRENCHES 8 TH... AND 42, 44 AND 45
4. SEE SHEETS ... AND ... FOR LEGENDS
5. ALL SOIL CLASSIFICATIONS SHOWN ON THIS SHEET AR...
6. TEST TRENCHES 9 THROUGH 14, 4..., 44 THRO... WERE EXCAVATED WITH A CASE 580 ... BACKH...
7. FINAL DEPTHS OF TEST TRENCHES 47 THROUGH 52 WER... IN BEDROCK OR COBBLES AND BOULDERS
8. THE SOFT BEDROCK PROFILE WAS DEVELOPED FROM ... TEST TRENCHES
9. THE HARD BEDROCK PROFILE WAS DEVELOPED FROM ... TTB... 2
10. SEE SHEET ... FOR LOGS OF TEST TRENCHES 8... 44
11. TEST TRENCHES 82-1 THROUGH 82-6 WERE EXCAVAT...
12. THE NORTH WALL OF EACH TEST TRENCH WAS MAPP... OF TEST TRENCHES 82-2 AND 82-... WHERE THE 82...

VALUE ENGINEERING PAYS

Left
Abutment

LOGS OF EMBANKMENT TEST TRENCHES

TEST TRENCH NO	DEPTH TO BEDROCK	TOTAL DEPTH	DIMENSIONS	BEARING	REMARKS
46	—	8'	18' LONG 2' WIDE	N 67° E	0.0-6.0' SILTY SAND GRAVELLY SILTY SAND 6.0-8.0' CALICHE CEMENTED COBBLES AND BOULDERS 1.5 TO 2' IN DIAMETER REFUSAL IN COBBLES AND BOULDERS
47	8	8.5'	12' LONG 2' WIDE	N 75° E	0.0-6.0 SILTY SAND GRAVELLY SILTY SAND 6.0-8.5 RED BROWN HIGHLY WEATHERED GRANITE REFUSAL IN GRANITE AT 8.5
48	6.5	7'	16' LONG 2' WIDE	N 74° E	0.0-6.5' SILTY SAND GRAVELLY SILTY SAND 6.5-7.0' RED-BROWN DECOMPOSED GRANITE REFUSAL IN GRANITE AT 7.0
49	5.5	6'	22' LONG 2' WIDE	N 86° E	0.0-5.5 SANDY SILT TO SILTY SAND FEW GRAVELS 5.5-6.0' RED-BROWN HIGHLY WEATHERED GRANITE REFUSAL IN GRANITE AT 6
50	7.0	7'	22' LONG 2' WIDE	N 72° E	0.0-7.0' SANDY SILT TO SILTY SAND WITH SOME GRAVELS REFUSAL IN RED BROWN DECOMPOSED GRANITE AT 7.0'
51	—	8.5'	26 LONG 2 WIDE	N 84° E	0.0-7.0' SILTY SAND TO SANDY SILT WITH SOME GRAVELS 7.0-8.5 WELL CEMENTED COBBLES UP TO 6" IN DIAMETER REFUSAL IN COBBLES AT 8.5'
52	—	10.0'	25 LONG 2 WIDE	N 78° E	0.0-8.0' SANDY SILT TO SILTY SAND POORLY CONSOLIDATED 8.4-10.0' SANDY SILT POORLY COMPACTED REFUSAL IN CALICHE CEMENTED COBBLES AT 10'
53	—	8'	25' LONG 2 WIDE	N 75° E	0.0-7.0' SANDY SILT TO SILTY SAND 7.0-8.0 CALICHE-CEMENTED SANDY SILT REFUSAL IN CALICHE CEMENTED COBBLES AT 8
54	—	9'	25 LONG 2 WIDE	N 87° E	0.0-5.0' SANDY SILT TO SILTY SAND WITH SOME GRAVELS 5.0-8.0' POORLY CEMENTED COBBLES TO 1' DIAMETER WITH MINOR CALICHE REFUSAL IN COBBLES AT 9



PROFILE ALONG ℄ OUTLET WORKS

HORIZ SCALE IN FEET

VERT SCALE IN FEET

NOTES

1. SEE ~~SHEET 2~~ FOR THE GEOLOGIC MAP AND DIAMOND DRILL HOLE PLAN OF EXPLORATION
2. SEE ~~SHEET 9~~ FOR LOGS OF TEST TRENCHES 9 THROUGH 14
3. SEE ~~SHEETS 10~~ FOR LOGS OF TEST TRENCHES 81 THROUGH 82 AND 42 44 AND 45
4. SEE ~~SHEETS 4 AND 6~~ FOR LEGENDS
5. ALL SOIL CLASSIFICATIONS SHOWN ON THIS SHEET ARE VISUAL
6. TEST TRENCHES 9 THROUGH 14 42 44 THROUGH 54 AND 8 THROUGH 92 WERE EXCAVATED WITH A CASE 580 BACKHOE
7. FINAL DEPTHS OF TEST TRENCHES 46 THROUGH 54 WERE DETERMINED BY REFUSAL IN BEDROCK OR COBBLES AND BOULDERS
8. THE SOFT BEDROCK PROFILE WAS DEVELOPED FROM DIAMOND DRILL HOLES AND TEST TRENCHES
9. THE HARD BEDROCK PROFILE WAS DEVELOPED FROM DIAMOND DRILL HOLES AND TTB2 2
10. SEE ~~SHEET 8~~ FOR LOGS OF TEST TRENCHES 82 1 THROUGH 82 5
11. TEST TRENCHES 82 1 THROUGH 82 6 WERE EXCAVATED WITH A D9 DOZER
12. THE NORTH WALL OF EACH TEST TRENCH WAS MAPPED WITH THE EXCEPTION OF TEST TRENCHES 82 2 AND 82 WHERE THE WEST WALL WERE MAPPED

DATUM IS MEAN SEA LEVEL

SYMBOL	DESCRIPTION	DATE	APPROVAL
A	Revised Downstream alignment of outlet works	July 83	

REVISIONS

U S ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY: R.L.T

DRAWN BY

CHECKED BY

GILA RIVER BASIN

PHOENIX ARIZONA AND VICINITY (INCLUDING NEW RIVER)

NEW RIVER DAM
DAM EMBANKMENT (LEFT ABUTMENT)
AND OUTLET WORKS
PLAN AND PROFILE OF
TEST TRENCH EXPLORATION

SUBMITTED BY

DATE APPROVED

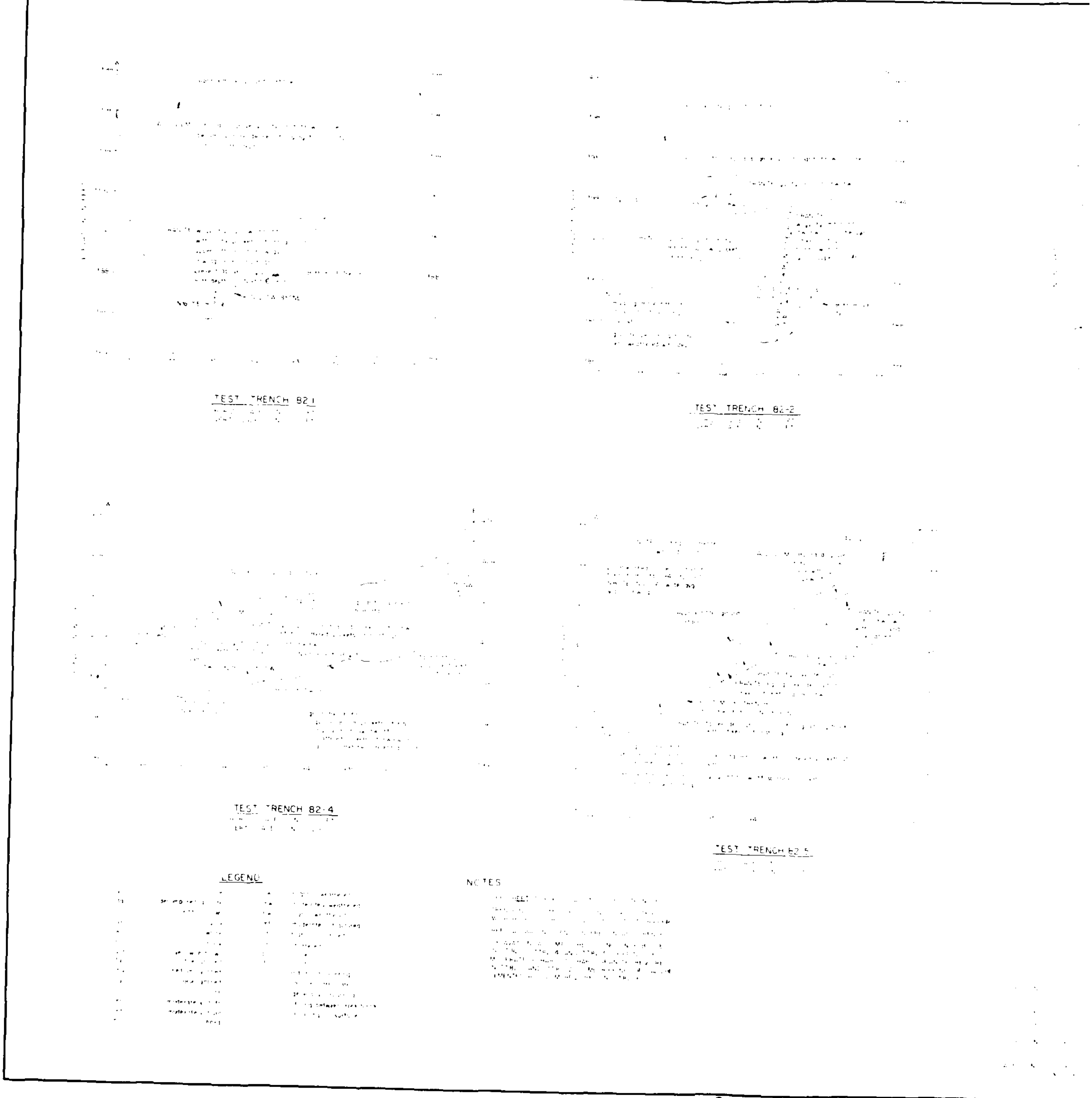
SPEC NO DACW09-

DISTRICT FILE NO 241/232

REV "A"

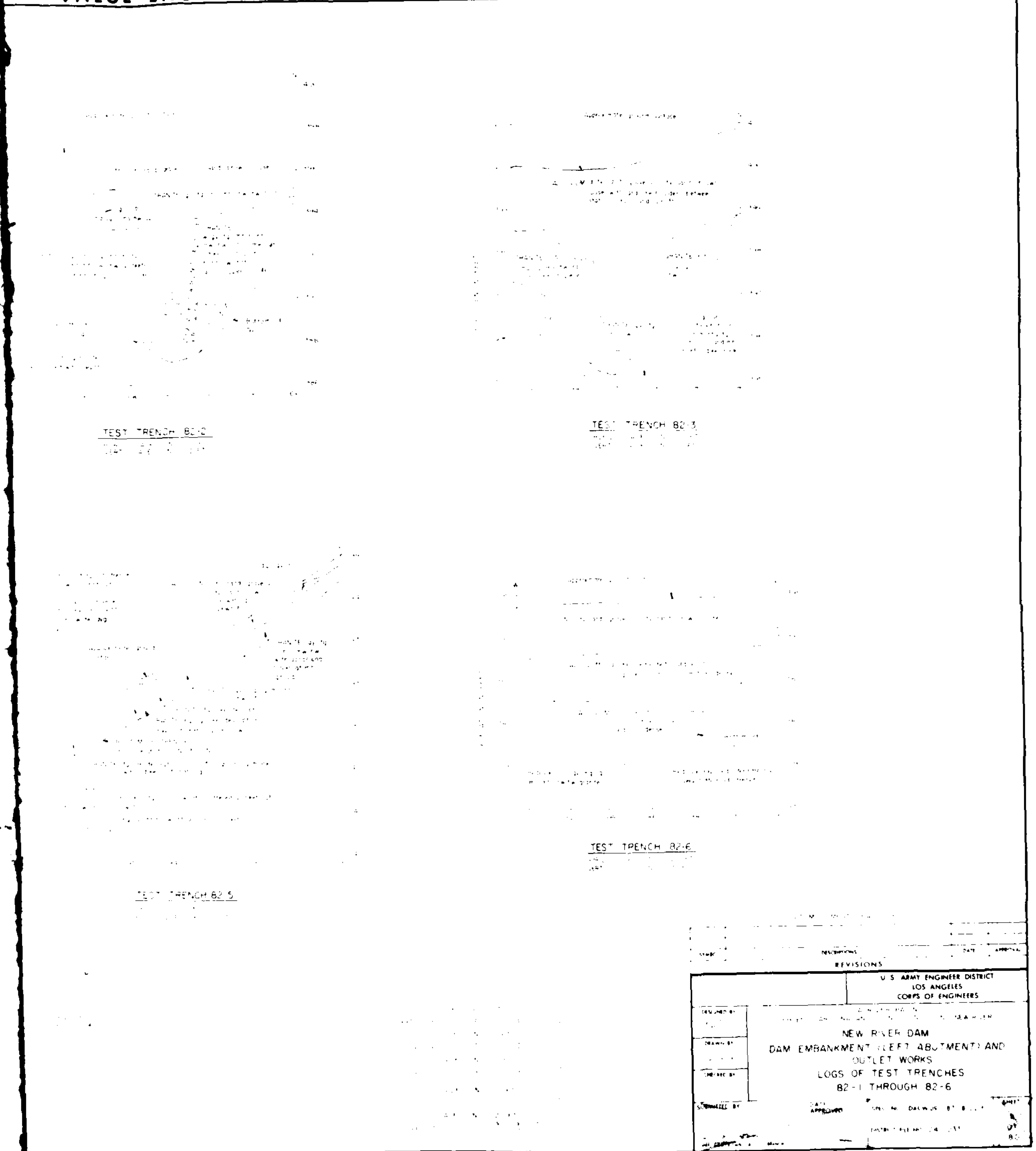
SAFETY PAYS

VALUE ENGINEERING PAYS



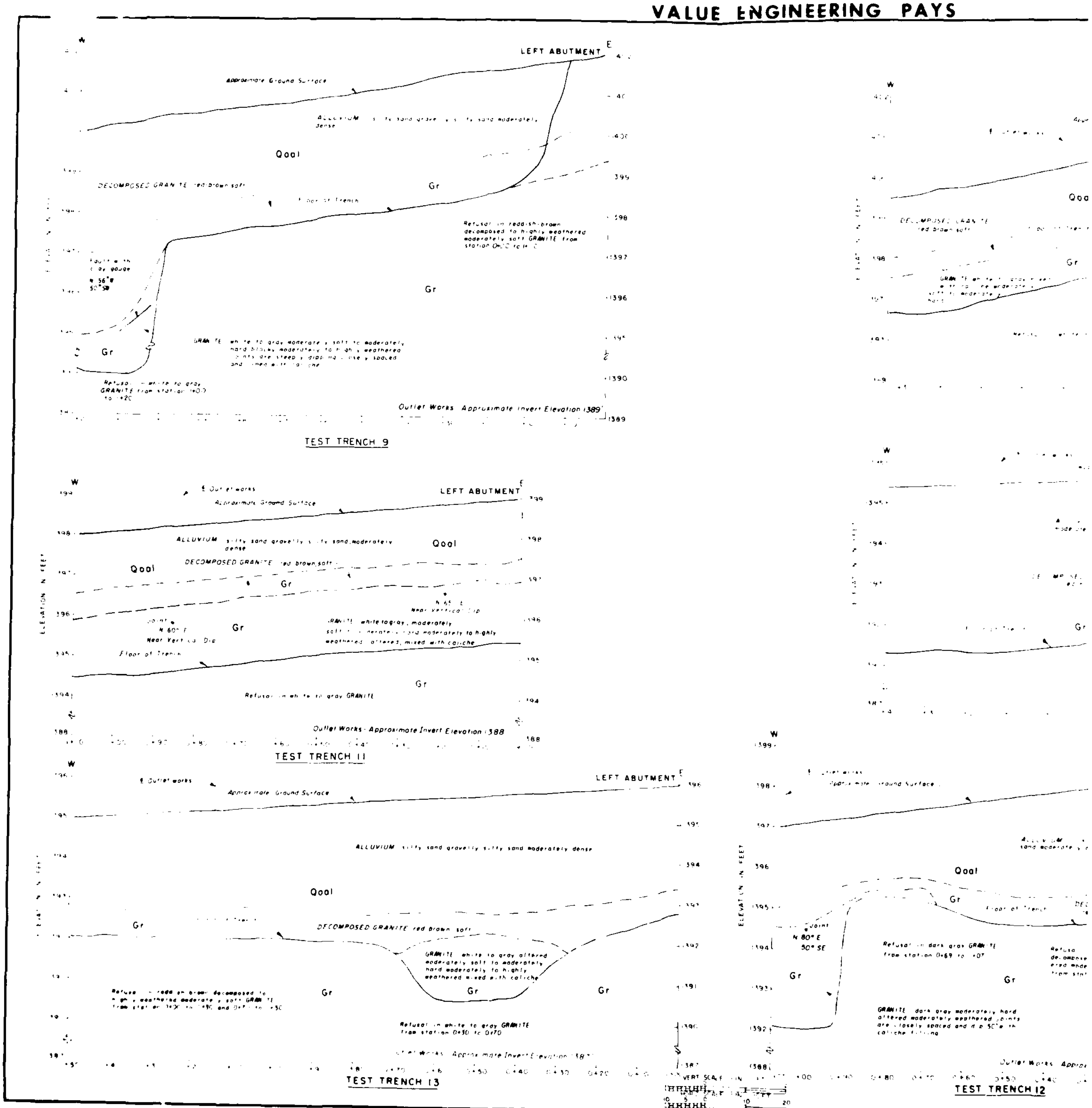
SAFETY PAYS

VALUE ENGINEERING PAYS



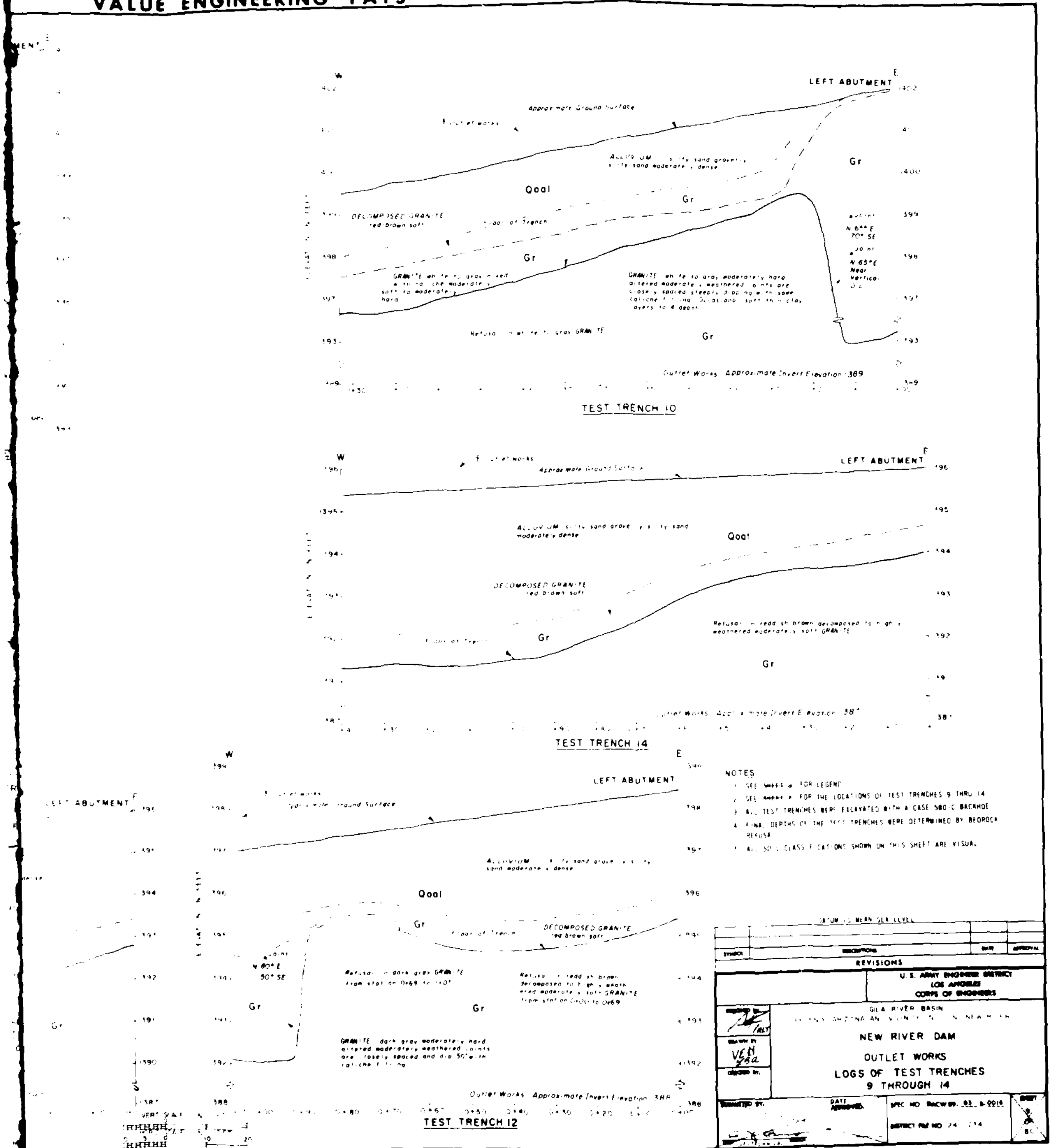
SAFETY PAYS

VALUE ENGINEERING PAYS



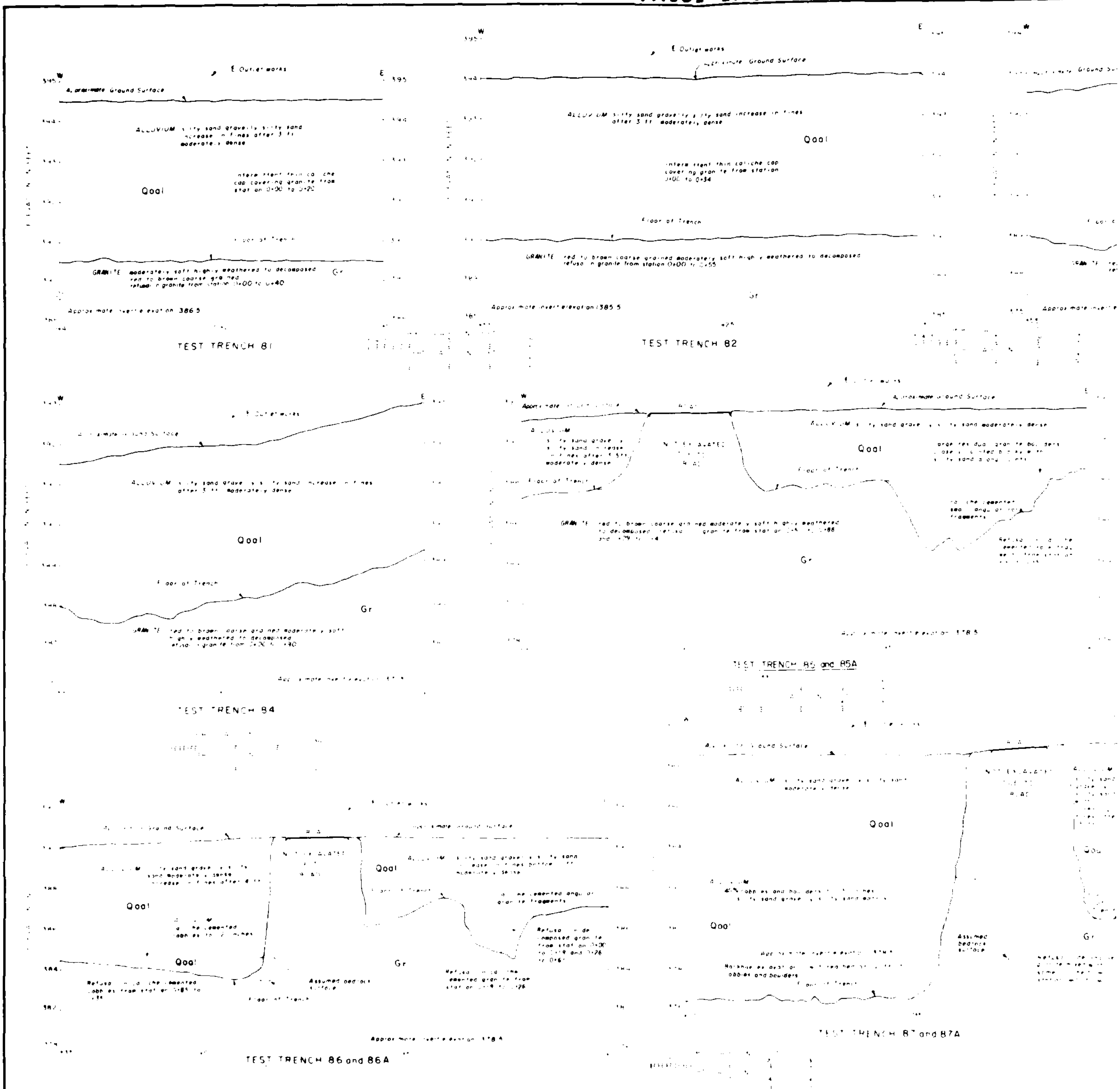
SAFETY PAYS

VALUE ENGINEERING PAYS



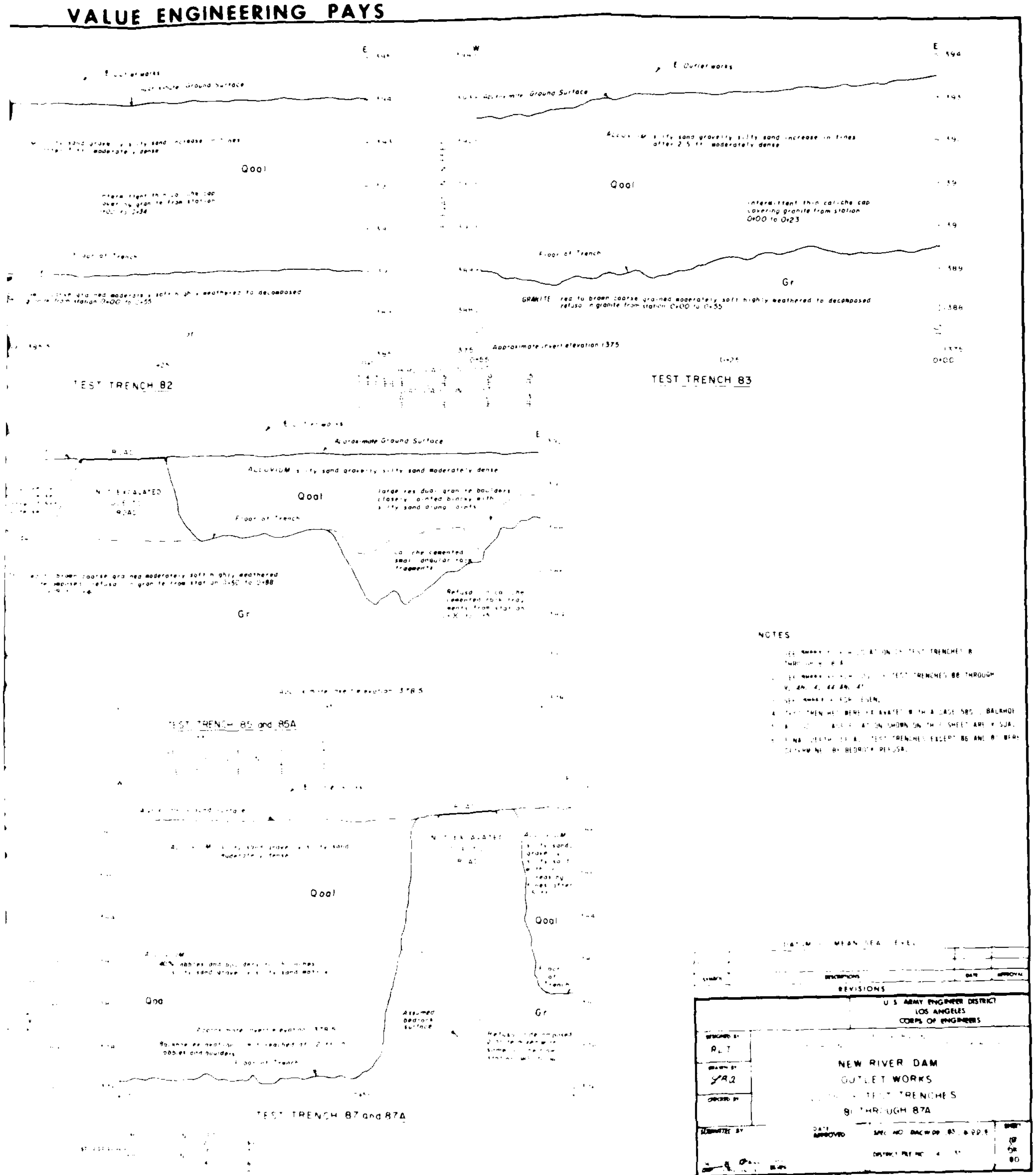
SAFETY PAYS

VALUE ENGINEERING PAYS

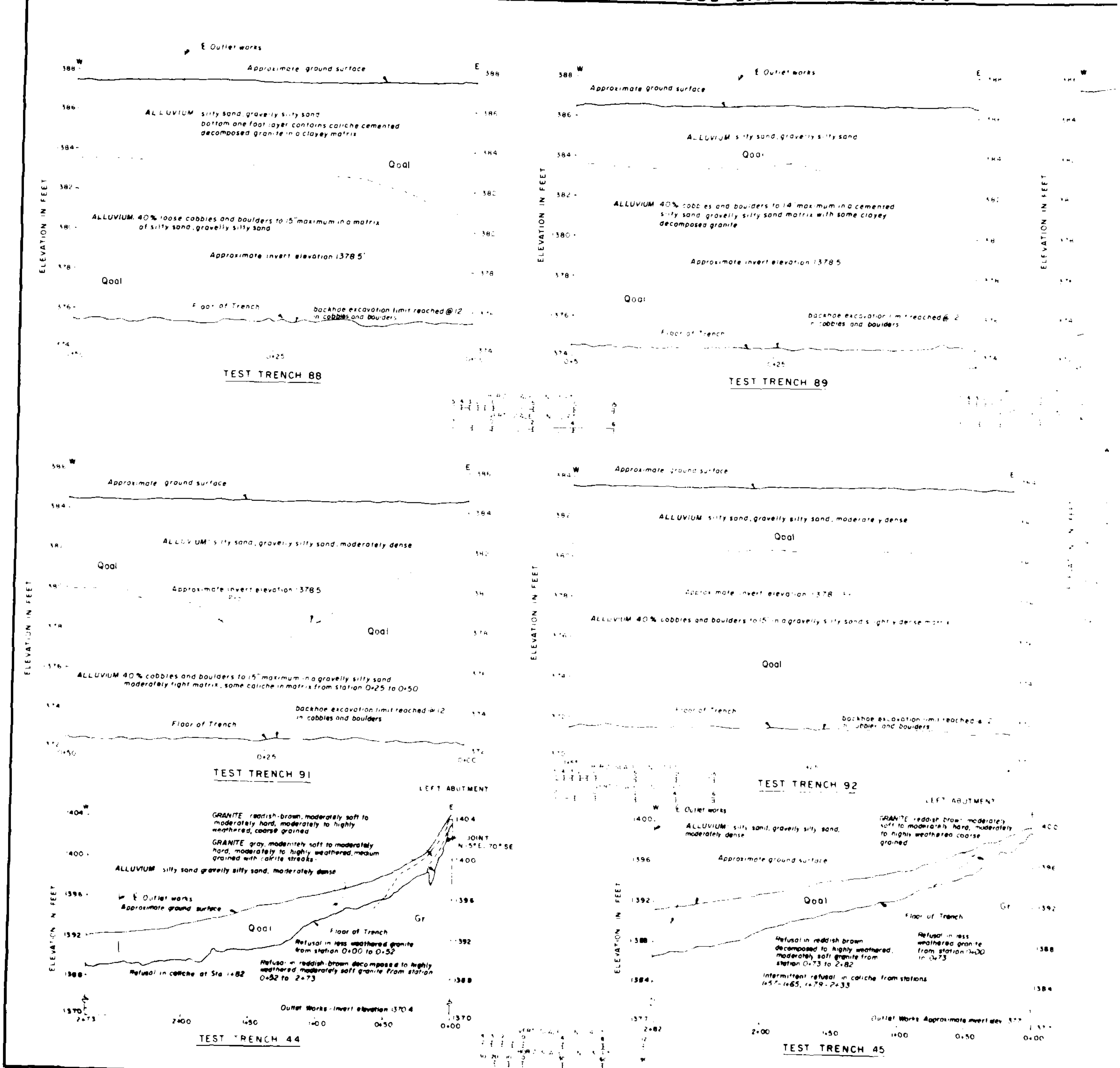


SAFETY PAYS

VALUE ENGINEERING PAYS

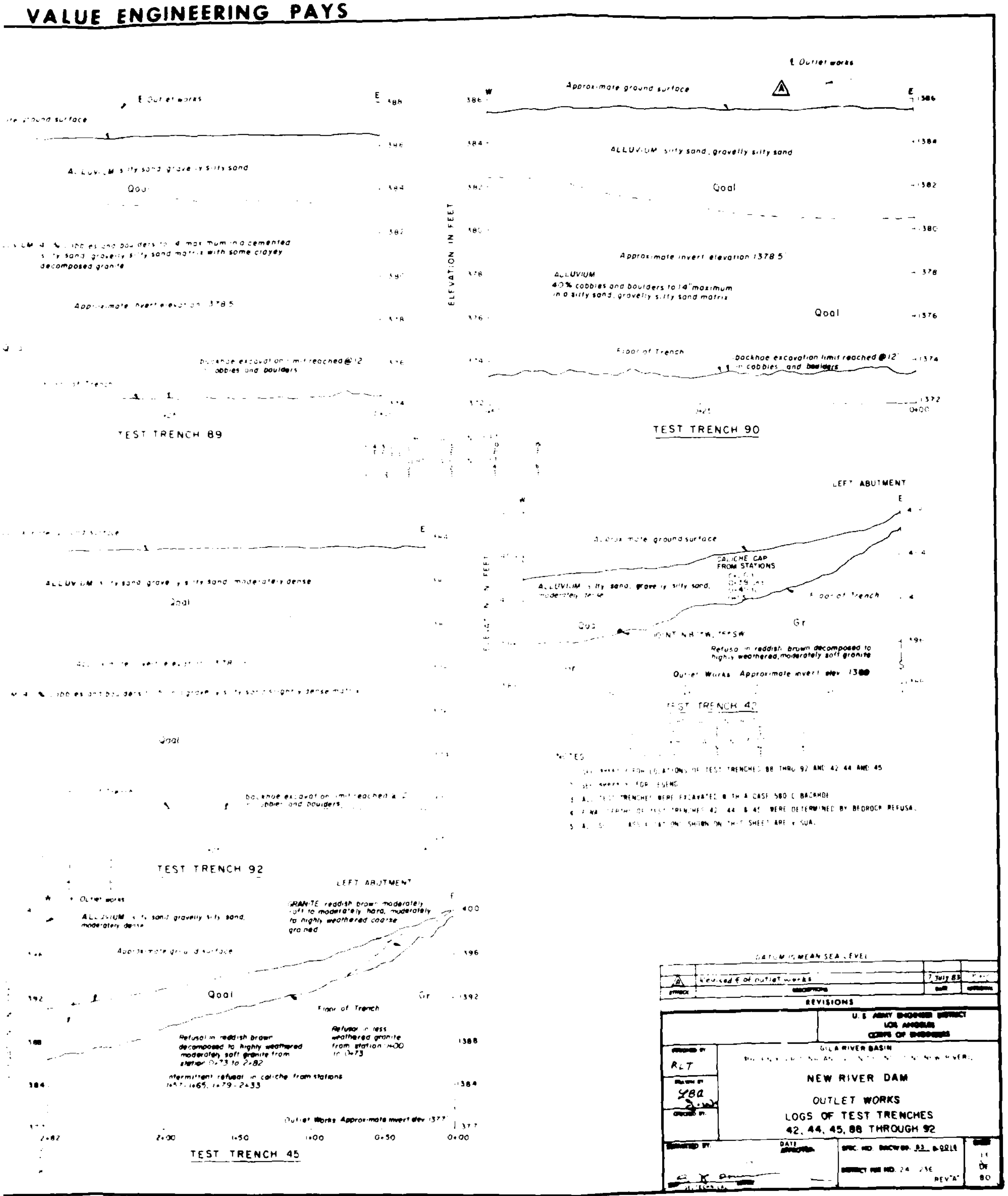


SAFETY PAYS

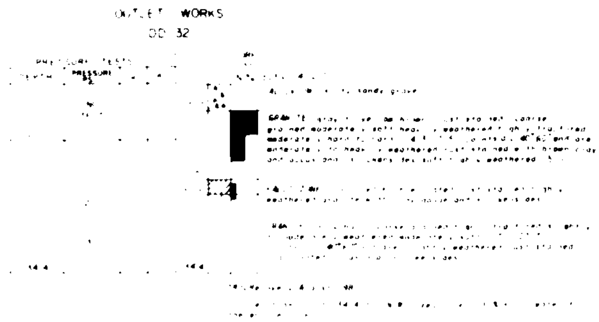


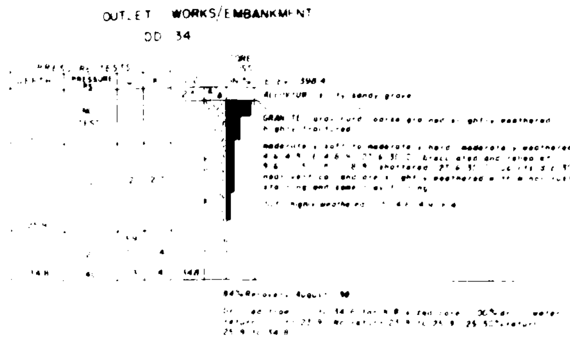
℄ Outlet works
Approximate ground surface
ALLUVIUM silty sand, gravelly silty sand bottom one foot layer contains caliche cemented decomposed granite in a clayey matrix
Qoal
ALLUVIUM 40% loose cobbles and boulders to 15" maximum in a matrix of silty sand, gravelly silty sand
Approximate invert elevation 1378.5'
Floor of Trench
backhoe excavation limit reached @ 12 in cobbles and boulders
ELEVATION IN FEET
0+25
TEST TRENCH 88
ALLUVIUM silty sand, gravelly silty sand
ALLUVIUM 40% cobbles and boulders to 14" maximum in a cemented silty sand, gravelly silty sand matrix with some clayey decomposed granite
Approximate invert elevation 1378.5
TEST TRENCH 89
ALLUVIUM silty sand, gravelly silty sand, moderately dense
Approximate invert elevation 1378.5
ALLUVIUM 40% cobbles and boulders to 15" maximum in a gravelly silty sand moderately tight matrix, some caliche in matrix from station 0+25 to 0+50
in cobbles and boulders
TEST TRENCH 91
ALLUVIUM 40% cobbles and boulders to 15" in a gravelly silty sand slightly dense matrix
TEST TRENCH 92
LEFT ABUTMENT
GRANITE reddish-brown, moderately soft to moderately hard, moderately to highly weathered, coarse grained
GRANITE gray, moderately soft to moderately hard, moderately to highly weathered, medium grained with calcite streaks
ALLUVIUM silty sand gravelly silty sand, moderately dense
JOINT N 5° E, 70° SE
Gr
Refusal in less weathered granite from station 0+00 to 0+52
Refusal in reddish-brown decomposed to highly weathered moderately soft granite from station 0+52 to 2+73
Refusal in caliche at Sta 1+82
Outlet Works - Invert elevation 1370.4
2+73
2+00
1+50
1+00
0+50
0+00
TEST TRENCH 44
ALLUVIUM silty sand, gravelly silty sand, moderately dense
GRANITE reddish brown, moderately soft to moderately hard, moderately to highly weathered coarse grained
Refusal in reddish brown decomposed to highly weathered, moderately soft granite from station 0+73 to 2+82
Refusal in less weathered granite from station 0+00 to 0+73
Intermittent refusal in caliche from stations 1+57-1+65, 1+79-2+33
Outlet Works Approximate invert elev.
2+82
TEST TRENCH 45

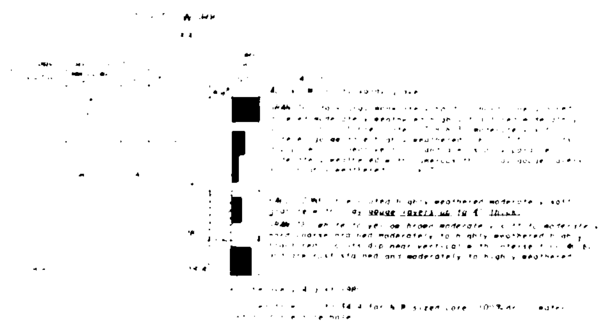
VALUE ENGINEERING PAYS

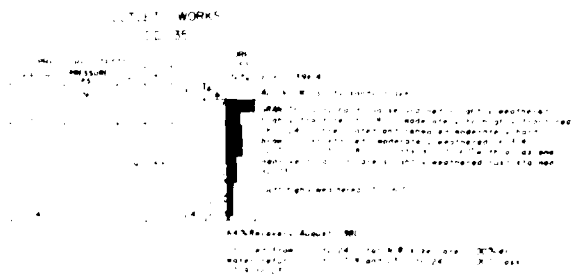


SAFETY PAYS


WORKS
DD 32

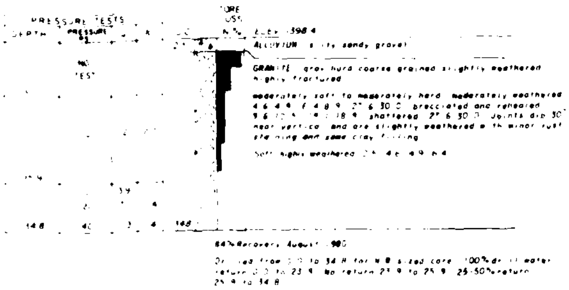

OUTLET WORKS/EMBANKMENT
DD 34




WORKS

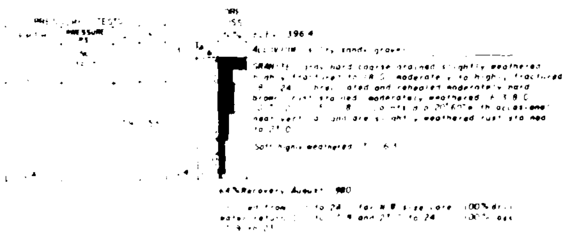
OUTLET WORKS/EMBANKMENT

DD - 34



OUTLET WORKS

DD - 35



REVISIONS

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

RIVER BASIN

DESIGNED BY RLT

DRAWN BY

CHECKED BY

SUBMITTED BY

APPROVED

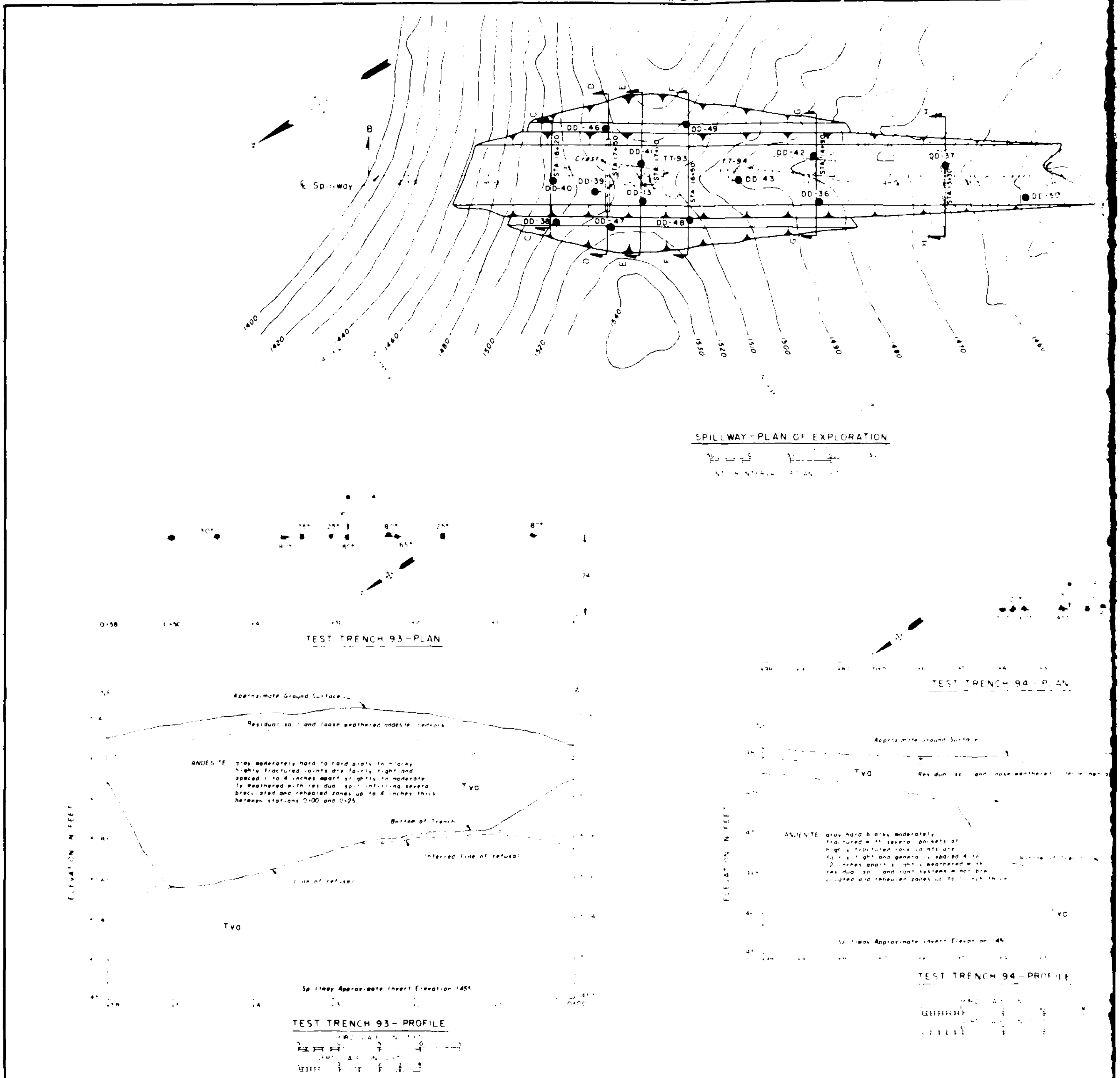
NEW RIVER DAM

OUTLET WORKS

GEOLOGIC LOGS OF DIAMOND DRILL HOLES

SHEET

VALUE ENGINEERING PAYS



SPILLWAY - PLAN OF EXPLORATION

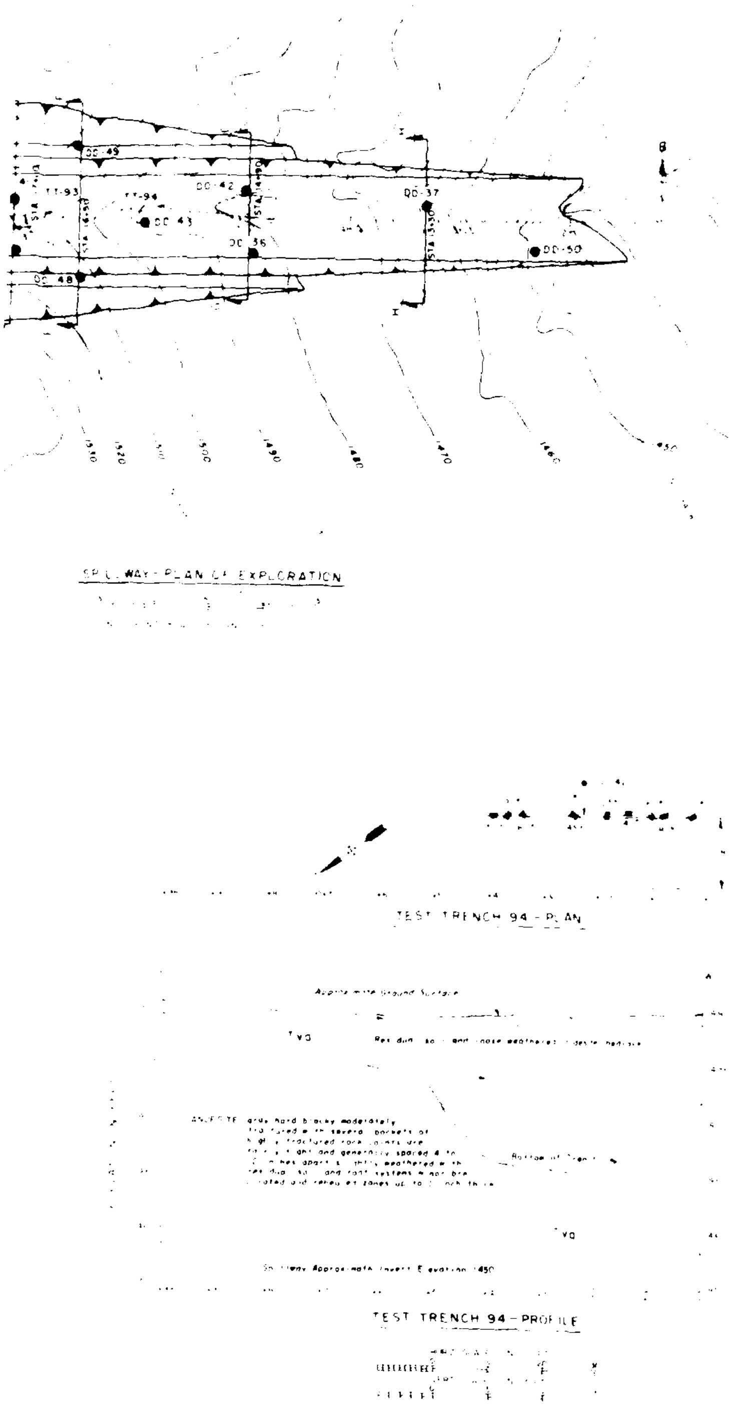
TEST TRENCH 93 - PLAN

TEST TRENCH 93 - PROFILE

TEST TRENCH 94 - PLAN

TEST TRENCH 94 - PROFILE

SAFETY PAYS



NOTES

SEE ... FOR SPILLWAY GEOLOGIC MAP AND LOGS AND LOCATIONS OF TEST TRENCHES 58 THROUGH 80

2. SEE SHEET ... FOR SPILLWAY PROFILE AND SECTIONS

3. SEE SHEET ... 95 FOR GEOLOGIC LOGS OF DIAMOND DRILL HOLES

4. SEE SHEET ... FOR LEGEND

5. PROFILES SHOWN ARE OF THE SOUTHEAST WALL OF EACH TRENCH

6. TRENCHING ACCOMPLISHED USING D 9G DOZER WITH HYDRAULIC BLADE AND SINGLE TOOTH RIPPER RIPPING FROM A SOUTHWEST TO NORTHEAST DIRECTION IN TEST TRENCH 93 AND FROM A NORTHEAST TO SOUTHWEST DIRECTION IN TEST TRENCH 94

7. NO LINE OF REFUSAL WAS ENCOUNTERED IN TEST TRENCH 94

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

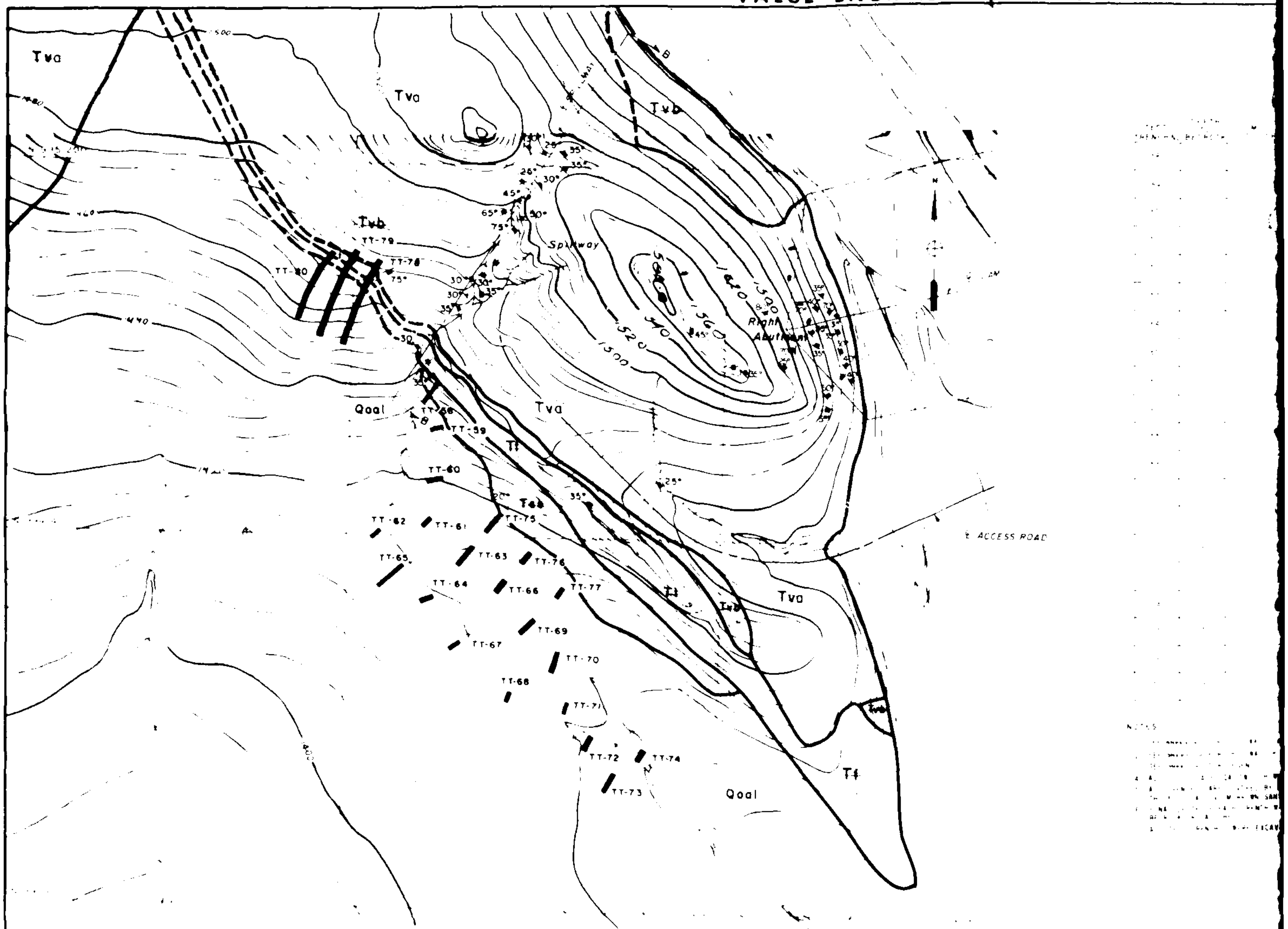
REVISIONS

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

NEW RIVER DAM

SPILLWAY PLAN OF EXPLORATION
AND LOGS OF TEST TRENCHES

DESIGNED BY
DRAWN BY
CHECKED BY
SUBMITTED BY
APPROVED
SPEC. NO. DACW09 - 83 - B-0016
DISTRICT FILE NO.
SHEET 15

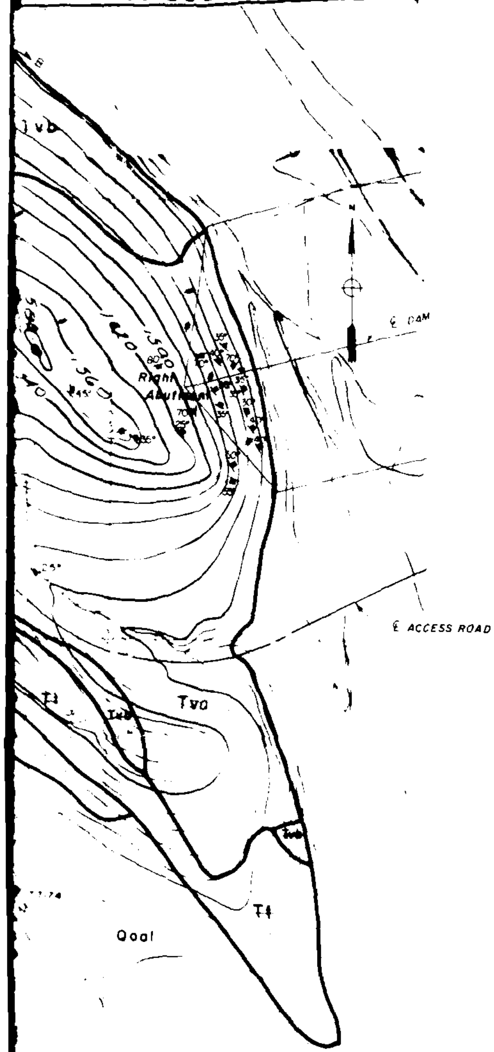


SPILLWAY TEST TRENCH PLAN

SPILLWAY TEST TRENCHES

[illegible] TRENCH NO.	DEPTH TO BEDROCK	MAXIMUM DEPTH	DIMENSIONS	BEARING	REMARKS: [illegible] LOG OF THE WEST WALL OF EACH TRENCH
78	[illegible]	7	[illegible] LONG 2' WIDE	N [illegible]°E	[illegible]
79	3.5	7	100' LONG 2' WIDE	N 29°E	[illegible]
80	3.5	5.5	40' LONG 2' WIDE	N 34°E	[illegible]

VALUE ENGINEERING PAYS



SPILLWAY TEST TRENCHES

TEST TRENCH No.	DEPTH TO BEDROCK	MAX DEPTH	DIMENSIONS	BEARING	REMARKS
58	0.0-1.0	2.0	48' LONG 2' WIDE	N 35 E	REFUSAL IN [illegible] SANDSTONE @ 2.0 [illegible] AND [illegible] N 55 [illegible]
59	1.5	2.0	28' LONG 2' WIDE	N 85 E	REFUSAL IN BROWN SANDSTONE @ 2
60	0.5	2.0	38' LONG 2' WIDE	[illegible]	REFUSAL IN SANDSTONE @ [illegible]
61	0.5	1.5	35' LONG 2' WIDE	N 40 E	REFUSAL IN BROWN SANDSTONE @ 1.5
62	0.5	1.0	25' LONG 2' WIDE	N 55 E	REFUSAL IN TUFF @ 1.0
63	1.0	2.5	55' LONG 2' WIDE	N 43 E	REFUSAL IN WHITE SANDSTONE @ [illegible] INTERMINGLED BROWN SANDSTONE LAYERS @ [illegible]
64	0.5	0.5	30' LONG 2' WIDE	N 70 E	REFUSAL IN TUFF @ 0.5
65	0.5	0.5	76' LONG 2' WIDE	N 47 E	REFUSAL IN CALICHE CEMENTED BRECCIA @ 0.5 IN THE S END OF THE TRENCH AND IN INTERMINGLED TUFF AND CALICHE TOWARD THE N END
66	0.5	2.0	35' LONG 2' WIDE	N 40 E	REFUSAL IN BROWN SANDSTONE @ 2.0
67	0.5	0.5	31' LONG 2' WIDE	N 54 E	REFUSAL IN TUFF @ 0.5
68	0.5	0.5	30' LONG 2' WIDE	N 22 E	REFUSAL IN TUFF @ 0.5
69	1.0	1.5	40' LONG 2' WIDE	N 50 E	REFUSAL IN BROWN TUFF @ 1.5
70	1.0	1.0	47' LONG 2' WIDE	N 12 E	REFUSAL @ 1 FT IN BROWN SANDSTONE [illegible] FROM THE N END OF THE TRENCH FROM [illegible] 47' REFUSAL @ [illegible] FT IN TUFF
71	0.5	0.5	30' LONG 2' WIDE	N 12 E	REFUSAL IN TUFF @ 0.5
72	0.5	1.0	38' LONG 2' WIDE	N 21 E	REFUSAL IN TUFF @ 1.0
73	0.5	1.0	50' LONG 2' WIDE	N 28 E	REFUSAL IN TUFF @ 1.0
74	0.0-1.5	4.0	34' LONG 2' WIDE	N 42 E	REFUSAL @ [illegible] FT IN TUFF @ S END OF TRENCH REFUSAL @ 4 FT IN THE REST OF THE TRENCH IN WHITE GRAY SANDSTONE [illegible] TUFF [illegible]
75	0.0-0.5	1.0	44' LONG 2' WIDE	N 35 E	BROWN SANDSTONE OUTCROPS @ THE N END OF THE TRENCH [illegible]
76	1.5	3.5	37' LONG 2' WIDE	N 43 E	REFUSAL @ [illegible] FT IN BROWN SANDSTONE WHICH GRADES INTO WHITE GREY SANDSTONE [illegible] FROM THE N END OF THE TRENCH
77	0.5	1.5	34' LONG 2' WIDE	N 40 E	REFUSAL @ [illegible] FT IN BROWN SANDSTONE WHICH GRADES INTO WHITE GREY SANDSTONE [illegible] FROM THE N END OF THE TRENCH

NOTES

1. SEE ~~SHEET~~ FOR SPILLWAY PLAN OF EXPLORATION
2. SEE ~~SHEET~~ FOR SPILLWAY PROFILE AND SECTIONS
3. SEE ~~SHEET~~ FOR LEGEND
4. ALL SOIL CLASSIFICATIONS SHOWN ON THIS SHEET ARE VISUAL
5. ALL TRENCHES ARE COVERED BY A THIN LAYER (UP TO 1.5 FT THICK) OF ALLUVIUM BROWN SANDY SILT AND SLOPEWASH
6. FINAL DEPTH OF EACH TRENCH WAS DETERMINED BY REFUSAL IN BEDROCK OR CALICHE
7. ALL TEST TRENCHES WERE EXCAVATED WITH A CASE 580 [illegible] BACKHOE

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

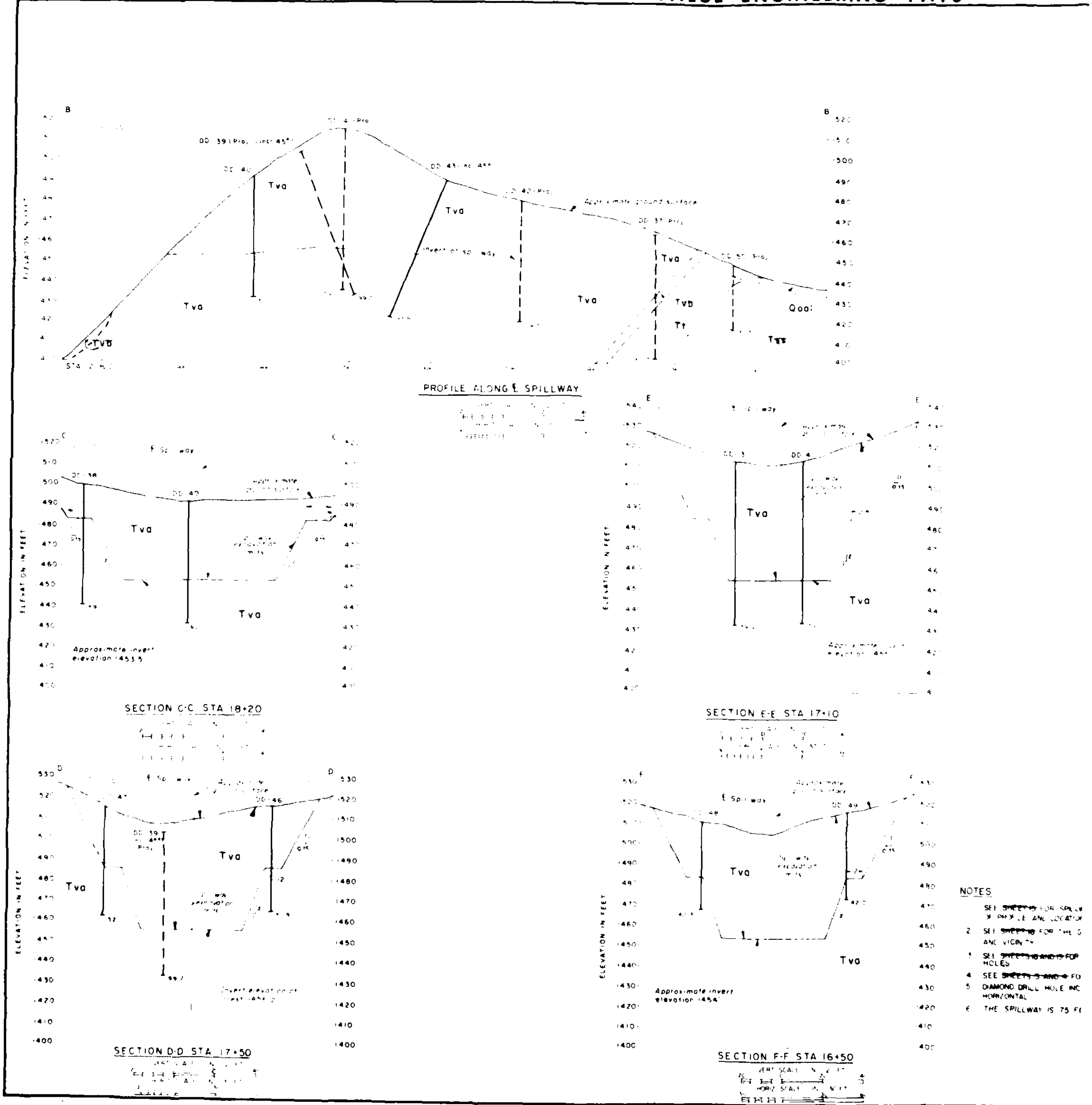
DESIGNED BY: RJT
DRAWN BY: YBa
CHECKED BY:

NEW RIVER DAM
SPILLWAY AND DAM EMBANKMENT
(RIGHT ABUTMENT) GEOLOGY
LOCATION AND LOGS OF
SPILLWAY TEST TRENCHES

SUBMITTED BY
DATE APPROVED
SPEC NO DACW09 83 B 0016
DISTRICT FILE NO 24 124
SHEET

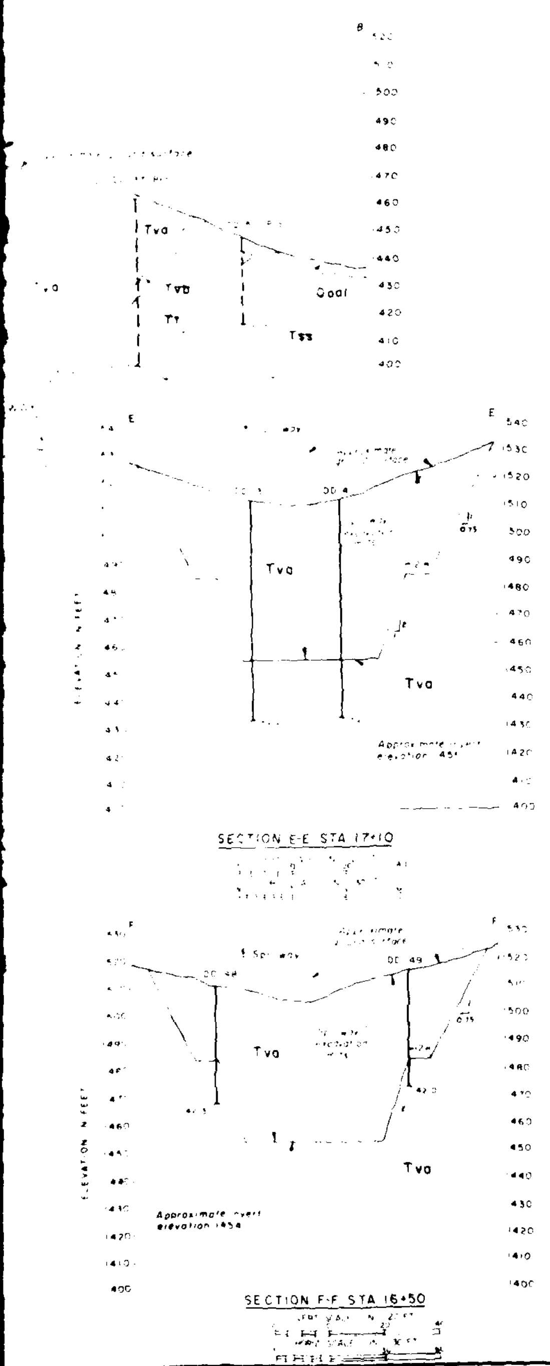
SAFETY PAYS

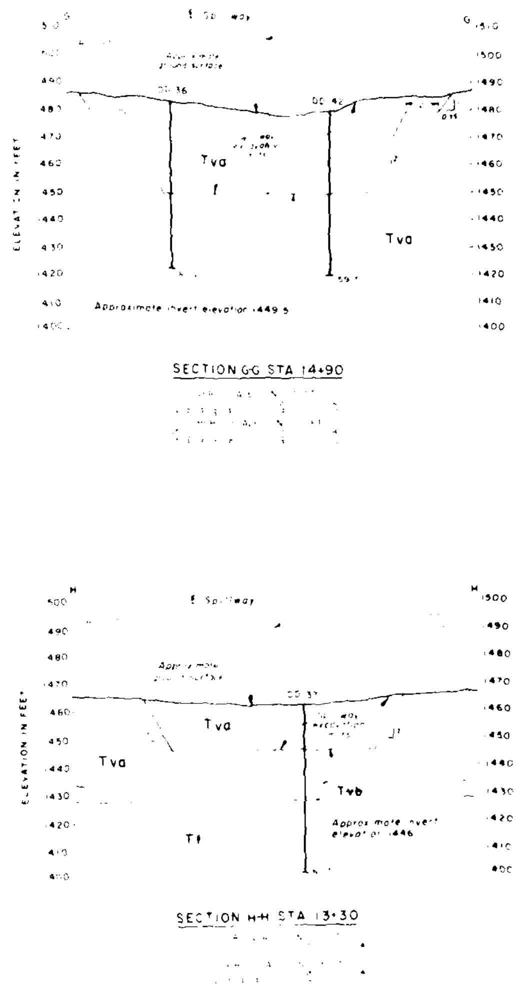
VALUE ENGINEERING PAYS



SAFETY PAYS

VALUE ENGINEERING PAYS





NOTES

1. SEE ~~SHEET 10~~ FOR SPILLWAY PLAN ... OF PROFILE AND LOCATION OF SECTIONS
2. SEE ~~SHEET 10~~ FOR THE GEOLOGIC MAP OF THE SPILLWAY SITE AND VICINITY
3. SEE ~~SHEETS 18 AND 19~~ FOR THE GEOLOGIC LOGS OF DIAMOND DRILL HOLES
4. SEE ~~SHEETS 3 AND 4~~ FOR LEGENDS
5. DIAMOND DRILL HOLE INCLINATIONS ARE MEASURED FROM HORIZONTAL
6. THE SPILLWAY IS 75 FEET WIDE AT THE BASE

DATUM IS MEAN SEA LEVEL

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY: F.L.T.
DRAWN BY: Y.R.G.
CHECKED BY:

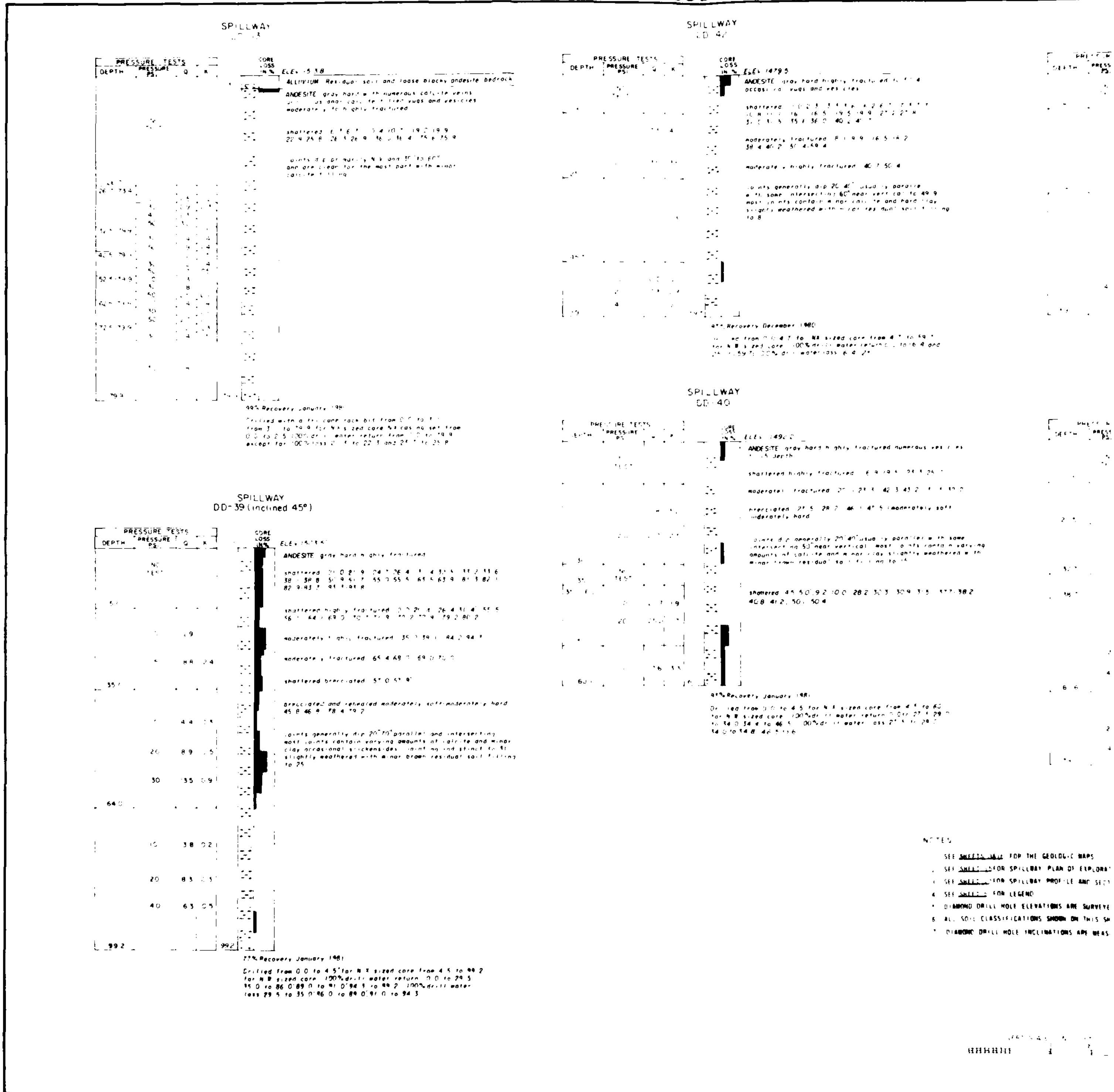
NEW RIVER DAM
SPILLWAY
PROFILE AND SECTIONS

SUBMITTED BY
DATE
APPROVED

SPEC. NO. DACW09-83-B-0015
DISTRICT FILE NO. 241/242
SHEET 17 OF 80

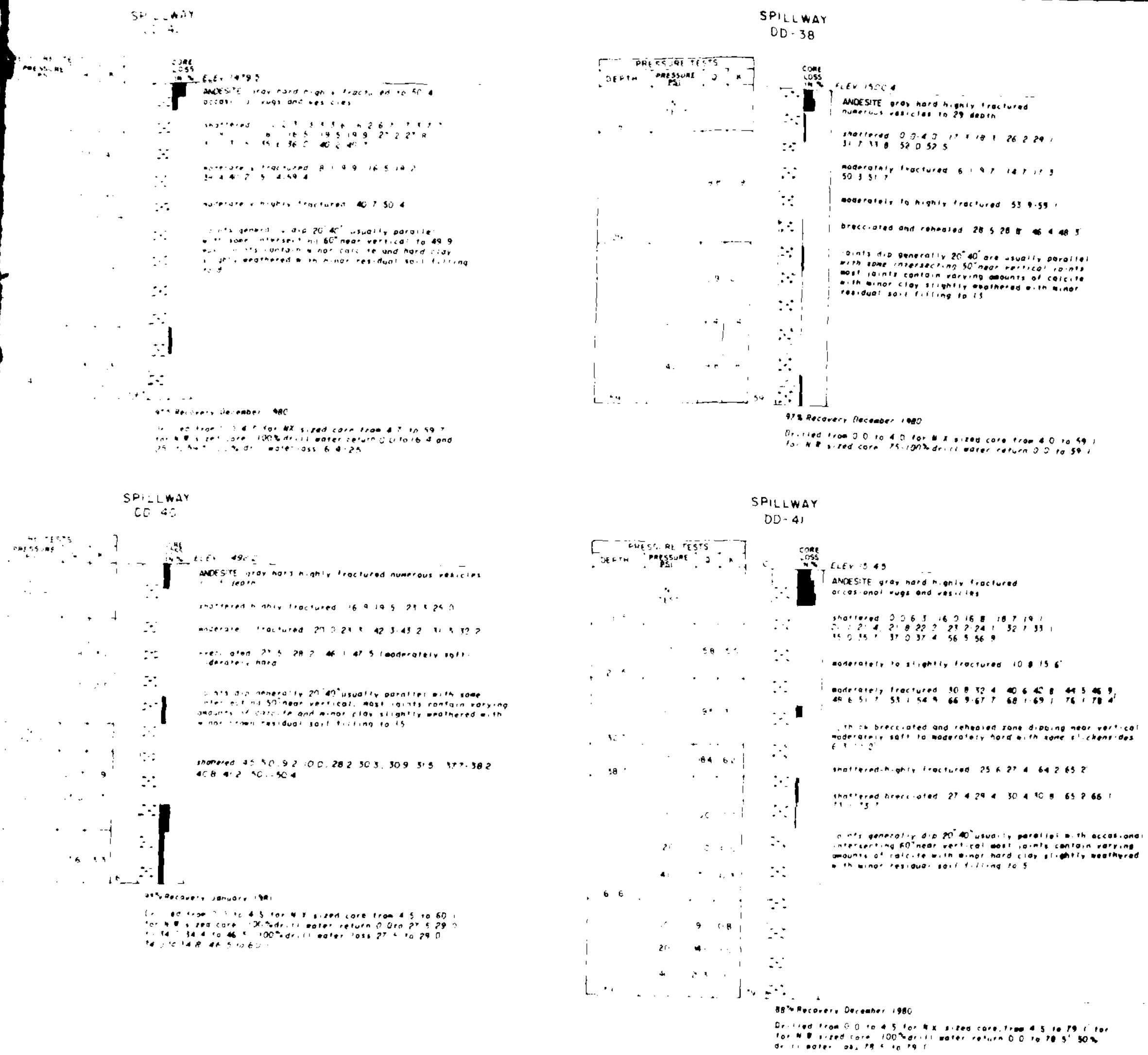
SAFETY PAYS

VALUE ENGINEERING PAYS



SAFETY PAYS

VALUE ENGINEERING PAYS



NOTES

1. SEE SHEETS FOR THE GEOLOGIC MAPS
2. SEE SHEET FOR SPILLWAY PLAN OF EXPLORATION
3. SEE SHEET FOR SPILLWAY PROFILE AND SECTIONS
4. SEE SHEET FOR LEGEND
5. DIAMOND DRILL HOLE ELEVATIONS ARE SURVEYED
6. ALL SOIL CLASSIFICATIONS SHOWN ON THIS SHEET ARE VISUAL
7. DIAMOND DRILL HOLE INCLINATIONS ARE MEASURED FROM HORIZONTAL

REVISIONS

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

NEW RIVER DAM
SPILLWAY
GEOLOGIC LOGS OF
DIAMOND DRILL HOLES
13, 38 THROUGH 42

DRAWN BY M.P.

DISTRICT FILE NO. 24/243

SAFETY PAYS

SPILLWAY

DD-43 (inclined 45°)

PRESSURE TESTS DEPTH	PRESSURE PSI	Q
NO TEST		
24		
	10	6.3
	20	9.4 1.2
39		
	10	3.1 0.3
	20	7.4 0.6
	30	0.4 0.7
59		
	0	5.7 0.5
	20	8.4 0.6
	40	2.7 0.6
77.8		
	0	3.6 0.3
	20	6.9 0.6
93.8		

CORE LOSS IN % ELEV 488.4

ANDESITE gray hard mostly highly fractured with occasional vugs and vesicles

shattered 0.0-0.8, 12.0-12.5, 24.6-24.9, 29.1-30, 42.0-42.6, 69.2-69.6, 74.4-74.8, 85.1-85.8, 90.8-91.3, 92.4-92.7

highly fractured to shattered 76.4-77.4

moderately to highly fractured 17.9-19.1, 24.1-34.1, 39.1-44.2, 49.2-54.2, 59.0-63.7, 64.0-66.5, 78.7-93.8

slightly fractured 77.4-78.7, 35.4-36.5

brecciated and rehealed 14.4-15.8 (moderately soft to moderately hard) 20.6-21.2, 24.1-24.9, 29.6-30.7, 31.4-32.1, 32.9-34.0, 43.6-44.1 (moderately soft)

joints generally dip 20°-70° mostly parallel with some intersecting 70° near vertical and contain varying amounts of calcite with minor amounts of red clay slightly weathered with some residual soil filling to 29

93.8'

99% Core Recovery December 1980

Drilled from 0.0 to 3.9 for NX sized core from 3.9 to 93.8 for NW sized core. 100% drill water return 0.0-8.7 intermittent 25-100% losses from 8.7 to 39. 100% loss from 39.1 to 93.8'

SPILLWAY

DD-46

PRESSURE TESTS DEPTH	PRESSURE PSI	Q	K
NO TEST			
21.2			
	5	0.0	0.0
	10	0.0	0.0
	20	2.4	0.4
32.2			
	5	12.6	1.8
	10	24.0	2.8
51.9			

CORE LOSS IN % ELEV 1516.4

2.2

ANDESITE gray hard highly fractured numerous vesicles to 10 depth

shattered 11.5-12.1, 38.9-39.3, 48.8-49.2

moderately fractured 28.0-29.6

joints generally dip 20°-40° mostly parallel with some intersecting 50°-70° slightly weathered with some residual soil filling to 14. most joints contain varying amounts of calcite with minor amounts of reddish-brown clay

51.9'

99% Recovery January 1981

Drilled from 0.0 to 2.2 with tri-cone rockbit. Drilled from 2.2 to 51.9 for NW sized core. 100% drill water return 0.0 to 49.0, 100% loss 49.0 to 51.9'

SPILLWAY

DD-49

PRESSURE TEST DEPTH	PRESSURE PSI	Q	K
NO TEST			
2.0			
	10	10.5	1.4
	20	15.4	5
	30	18.8	1.4
42.0			

CORE LOSS IN % ELEV 1516.2

2.0

ANDESITE gray hard highly fractured occasional vugs and vesicles

shattered to brecciated 2.2-4.6, 5.9-7.3

shattered 4.6-5.9, 7.5-7.9, 36.0-36.7

highly fractured to shattered 38-38.4, 39.6-39.7, 15.4-16.1

moderately to highly fractured 12.3-15.8

moderately to slightly fractured 27.2-32.4

brecciated and rehealed 7.9-8.7, 17.4-17.9, 40.0-40.4

joints generally dip 20°-40° are parallel with occasional intersecting 50° near vertical. most joints lined with calcite with minor hard red clay slightly weathered with brown residual soil filling to 16.5

42.0

99% Recovery February 1981

Drilled from 0.0-2.0 using tri-cone rockbit. Drilled from 2.0-42.0 for NW sized core. 75% drill water return 0.0-37, 100% loss 37-40, 50-75% return 40.0-42.0

SPILLWAY

DD-4?

PRESSURE TEST DEPTH	PRESSURE PSI	Q	K
NO TEST			
21.3			
	10	7	0
	20	9.7	0.9
	30	3.4	0.9
42.3			

CORE LOSS IN % ELEV 1510.9

2.5

ANDESITE gray hard moderately to highly fractured occasional vugs and vesicles

shattered 7.2-7.7, 19.1-18.6

slightly fractured (good solid rock) 25.3-26.9, 29.7-31.3, 32.3-33.1, 37.1-38.3, 39.1-40.3, 41.0-42.3

moderately to highly fractured 33.1-37.1

moderately fractured (one piece) 40.3-41.0

brecciated and rehealed 33.1-34.0

joints generally dip 20°-40° are parallel with some intersecting 50° near vertical. slightly weathered with some residual soil filling to 18.1. most joints contain varying amounts of calcite with minor reddish brown clay

42.3

99% Recovery February 1981

Drilled from 0.0-2.5 using tri-cone rockbit. Drilled from 2.5-42.3' for NW sized core. 100% drill water return 0.0-42.3

SPILLWAY

DD-5?

PRESSURE TEST DEPTH	PRESSURE PSI	Q	K
NO TEST			
0.5			
	10	0.1	0.02
	20	0.7	0
	30	1.0	0
31.5			

CORE LOSS IN % ELEV 44?6

0.0 4.5 7.4 14.7 31.5

ALLUVIUM residual soil and slopewash

TUFF pink moderately hard, fine grained with a few hard nearly aphanitic pink bands highly fractured moderately soft to moderately hard. 6.1-7.4 moderately soft brown highly weathered

TUFFACEOUS SANDSTONE dark brown moderately soft coarse grained poorly indurated with distinct layering, alternating coarser/finer grained bands (dipping 30°) scattered bands composed of pink tuff moderately to highly fractured 4.5-4.8'

SANDSTONE dark brown to gray, moderately soft fine grained, poorly indurated with layering (alternating coarser/finer grained bands) dipping 30°, moderately to slightly fractured

joints throughout core are moderately weathered coated with brown residual soil to 8.3, are clean, unweathered from 8.3-31.5' dip 30° are parallel with occasional intersecting 60°

100% Recovery February 1981

Drilled from 0.0 to 1.5 with auger. Drilled from 1.5-31.5 for NW sized core. 100% drill water return 0.0-31.5

SPILLWAY

DD-36

PRESSURE TEST DEPTH	PRESSURE PSI	Q	K
NO TEST			
9.2			
	10	0.0	0.0
	20	0.1	0.02
20.2			
	10	0.5	0.1
	20	1.6	0.2
	30	4.0	0.3
40.2			
	10	0.3	0.05
	20	1.0	0
	40	2.9	0.2
60.2			

CORE LOSS IN % ELEV 1483.3

ANDESITE gray hard highly fractured occasional vugs and vesicles

shattered 0.0-2.0, 35.4-36.0

moderately fractured 7.8-9.7, 55.2-58.2

moderately to highly fractured 20.2-25, 45.2-50.2

moderately to slightly fractured 50.2-50.7, 52.4-55.2

slightly fractured 45.4-46.6, 50.7-52.4

joints dip generally 20°-40°, usually parallel with intersecting 90° near vertical, joints slightly weathered with minor residual soil filling to 15

joints are either clean or contain minor calcite with some minor hard clay

60.2'

99% Recovery December 1980

Drilled from 0.0 to 4.2 for NX sized core, from 4.2 to 60.2 for NW sized core. 100% drill water return 0.0 to 60.2

NOTES

1. SEE ~~[illegible]~~ FOR THE ...
2. SEE ~~[illegible]~~ FOR ...
3. SEE ~~[illegible]~~ FOR ...
4. SEE ~~[illegible]~~ FOR LEGEND
5. DIAMOND DRILL HOLE ELEVATION ...
6. ALL SOIL CLASSIFICATIONS ...
7. DIAMOND DRILL HOLE IN ... HORIZONTAL

VALUE ENGINEERING PAYS

SPILLWAY

DD-46

PRESSURE TESTS

DEPTH	PRESSURE PSI	Q	K	CORE LOSS IN %
2.2				
	5	0.0	0.2	
	10	0.0	3.2	
	20	2.4	0.4	
32.2				
	5	2.6	1.8	
	10	24.0	2.8	
51.9				

NO TEST

ELEV 1516.4

ANDESITE gray hard highly fractured numerous vesicles to 10' depth

shattered 1.1 3.12.1 39.9 39.3 48.8 49.2

moderately fractured 28.0 29.6

joints generally dip 20°-40° mostly parallel with some intersecting 50°-70° slightly weathered with some residual soil filling to 14 most joints contain varying amounts of calcite with minor amounts of reddish brown clay

51.9

99% Recovery January 1981

Drilled from 0.0 to 2.2 with tri-cone rockbit. Drilled from 2.2 to 51.9 for NW sized core. 100% drill water return 0.0 to 49.0. 100% loss 49.0 to 51.9

SPILLWAY

PRESSURE TESTS

DEPTH	PRESSURE PSI	Q	K	CORE LOSS IN %
2.6				
21.1				
	10	4.4		
	20	5.7	0	
32.1				
	10	14.5		
	20	9.5	6	
52.1				

NO TEST

ELEV 1544

ANDESITE gray hard highly fractured with occasional vug and vesicles

shattered 5.5 5.8.6 1 6.8 7.0 7.5 12.6 19.5 22.1 22.6 46.4 47.0

shattered to highly fractured 20.4 2 6 24.6 27.2 33.5 37 47.6 49.4

moderately to slightly fractured 50.5 52.1

slightly fractured solid pieces 22.6 23.8 38.4 39.8

42.9 45.6 near vertical rough joint lined with calcite healed brecciated felsite up to ½" thick

joints generally dip 20°-40° mostly parallel with some intersecting 50° near vertical slightly weathered with minor brown residual soil filling to 12 most joints contain varying amounts of calcite with some minor hard reddish brown clay

52

100% Recovery February 1981

Drilled from 0.0 to 2.6 using tri-cone rockbit. Drilled from 2.6 to 52 for NW sized core. 100% drill water return 0.0 41.5. 100% loss 41.5 52.1

SPILLWAY

PRESSURE TESTS

DEPTH	PRESSURE PSI	Q	K	CORE LOSS IN %
6.7				
	10	2.3	0.4	
	20	5.2	0.5	
30.7				
	10	18.2	5	
	20	24.1	5	
60.7				

NO TEST

ELEV 1462.6

ANDESITE gray hard highly fractured

shattered 0.0 2.3 4.5 5.8 17.5 18.1

shattered to highly fractured 2.3 4.5

brecciated 7.6 8.0

joints generally dip 20°-40° usually parallel with intersecting 70° near vertical indistinct jointing to 4.5

ANDESITE purple gray hard highly fractured with near vertical dipping thin dikes of red punky moderately soft tuff moderately hard rehealed breccia at 26.2 26.4 26.8 27.2 28.4 28.6

reddish alteration to andesite at 27.2 28.5

FLOW BRECCIA reddish brown moderately hard to hard, moderately to highly fractured (no distinct joints) massive 27.8 29.4, contains angular hard purple-gray andesite fragments well cemented in a reddish brown partially clayey and vesicular matrix with scattered pockets of highly vesicular moderately soft to moderately hard scoria, occasional calcite stringers

brecciated contact between flow breccia and tuff

TUFF moderately hard to hard, moderately to highly fractured, pinkish brown to purple gray banded (bands dip 30°) glassy, fine to coarse grained

joints dip generally 30° are parallel with occasional intersecting 50°-60° joints joints are either clean or contain minor amounts of calcite, brownish clay with occasional slickensides

shattered 58.3 58.7

60.7

90% Recovery January 1981

Drilled from 0.0-4.5 for NX sized core, from 4.5-60.7 for NW sized core. 100% drill water return 0.0-42.7, 43.2-52.0. 100% water loss 42.7-43.2, 52.0-60.7

SPILLWAY

DD-49

PRESSURE TEST

DEPTH	PRESSURE PSI	Q	K	CORE LOSS IN %
2.0				
22.0				
	10	0.5	4	
	20	15.4	5	
	30	18.8	4	
42.0				

NO TEST

ELEV 1516.2

ANDESITE gray hard highly fractured occasional vugs and vesicles

shattered to brecciated 2.0 4.6 5.9 7.3

shattered 4.6 5.9 7.5 7.9 36.0 36.7

highly fractured to shattered 38.1 38.4 38.6 39.7 5.4 16.1

moderately to highly fractured 12.3 13.8

moderately to slightly fractured 27.2 32.4

brecciated and rehealed 7.9 8.7, 17.4 17.9 40.0 40.4

joints generally dip 20°-40° are parallel with occasional intersecting 50° near vertical most joints lined with calcite with minor hard red clay, slightly weathered with brown residual soil filling to 16.5

42.0

99% Recovery February 1981

Drilled from 0.0-2.0 using tri-cone rockbit. Drilled from 2.0-42.0 for NW sized core. 75% drill water return 0.0-37, 100% loss 37-40.50, 75% return 40.0-42.0

SPILLWAY

DD-36

PRESSURE TEST

DEPTH	PRESSURE PSI	Q	K	CORE LOSS IN %
4.2				
	10	0.0	0.0	
	20	0	0.02	
24.2				
	10	0.5	0	
	20	1.6	0.2	
	30	4.0	0.3	
41.2				
	10	0.3	0.03	
	20	1.0	0	
	40	2.9	0.2	
60.2				

NO TEST

ELEV 1483.3

ANDESITE gray, hard, highly fractured occasional vugs and vesicles

shattered 0.0 2.0 35.4 36.0

moderately fractured 7.8 9.7 55.2 58.2

moderately to highly fractured 20.2 25.1 45.2 50.2

moderately to slightly fractured 30.2 30.7 52.4 55.2

slightly fractured 45.4 46.6, 50.7 52.4

joints dip generally 20°-40°, usually parallel with intersecting 50° near vertical, joints slightly weathered with minor residual soil filling to 15

joints are either clean or contain minor calcite with some minor hard clay

60.2

99% Recovery December 1980

Drilled from 0.0 to 4.2 for NX sized core, from 4.2 to 60.2 for NW sized core. 100% drill water return 0.0 to 60.2

NOTES

1. SEE ~~SHEET~~ FOR THE GEOLOGIC MAPS
2. SEE ~~SHEET~~ FOR SPILLWAY PLAN OF EXPLORATION
3. SEE ~~SHEET~~ FOR SPILLWAY PROFILE AND SECTIONS
4. SEE ~~SHEET~~ FOR LEGEND
5. DIAMOND DRILL HOLE ELEVATIONS ARE SURVEYED
6. ALL SOIL CLASSIFICATIONS SHOWN ON THIS SHEET ARE VISUAL
7. DIAMOND DRILL HOLE INCLINATIONS ARE MEASURED FROM HORIZONTAL

VERT SCALE IN FEET

0 10 20 30

DAT: MEAN SEA LEVEL

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY: RLT

DRAWN BY: YAA

CHECKED BY:

GILA RIVER BASIN

NEW RIVER DAM
SPILLWAY
GEOLOGIC LOGS OF DIAMOND DRILL HOLES
36,37,43,46 THROUGH 50

SUBMITTED BY:

DATE APPROVED

SPEC NO. DACW09 83 B 0019

DISTRICT FILE NO 241/244

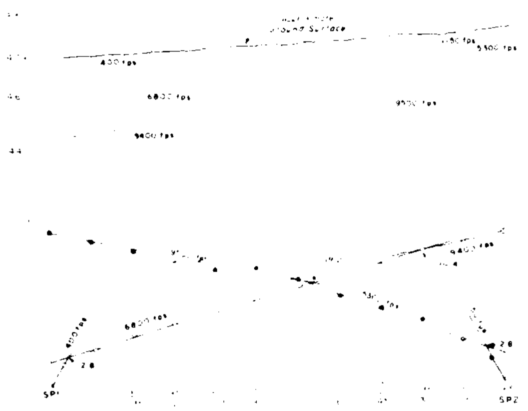
SHEET 19 OF 80

SAFETY PAYS

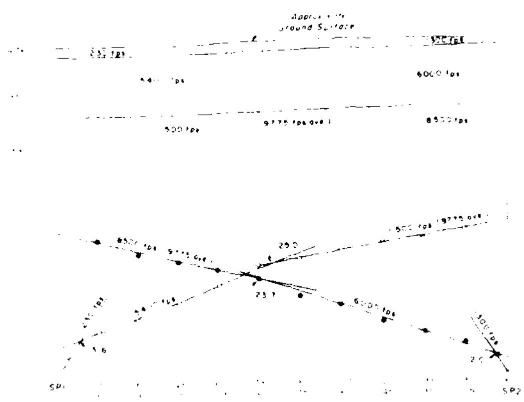
FOUNDATION REPORT

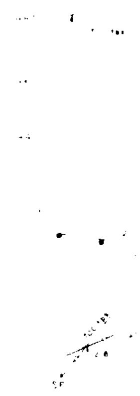
PLATE 22

DIKE NO 2 LINE 6

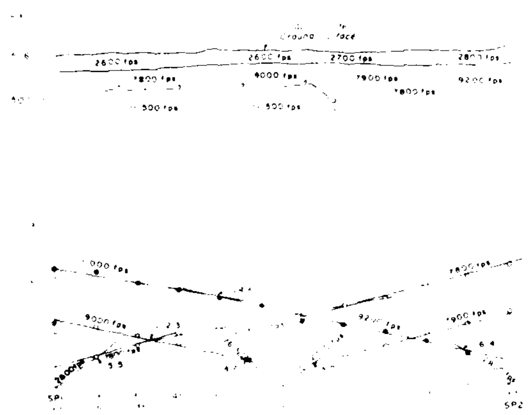


DIKE NO 2 LINE 7

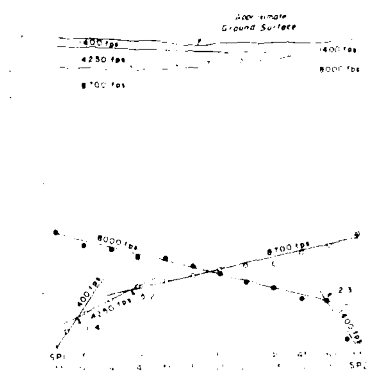


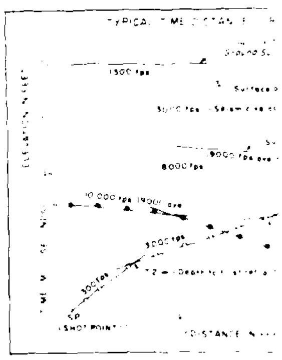


SPILLWAY LINE 9



SPILLWAY LINE 10



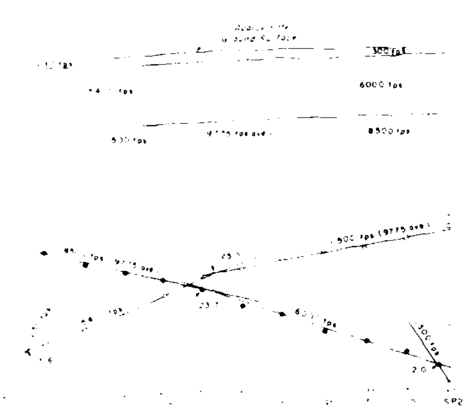


NOTES

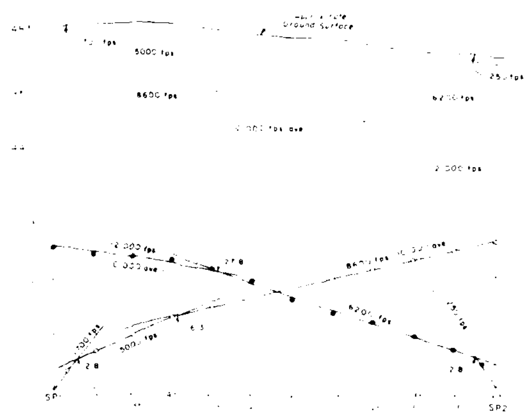
1. SEE SHEET 20 FOR APPROXIMATE LOCATIONS OF SEISMIC SURVEY.
2. SHOT POINT NUMBER 1 (SP1) FOR ANY LINE IS AT THE SOUTHERN END AND SHOT POINT NUMBER 2 (SP2) IS AT THE NORTHERN END.
3. REFRACTIVE SEISMIC SURVEY ACCOMPLISHED OCTOBER [illegible] BY LOS ANGELES DISTRICT PERSONNEL WITH A 12 CHANNEL [illegible] SEISMOGRAPH.
4. INTERPRETATION OF DATA FROM THE REFRACTIVE SEISMIC SURVEY WAS DONE BY LOS ANGELES DISTRICT PERSONNEL.

VALUE ENGINEERING PAYS

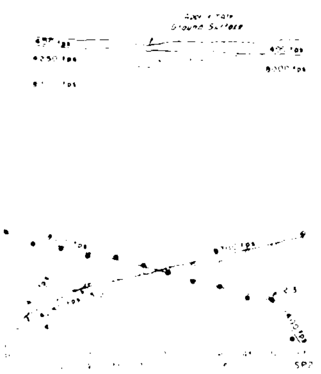
DIKE NO 2 LINE 7

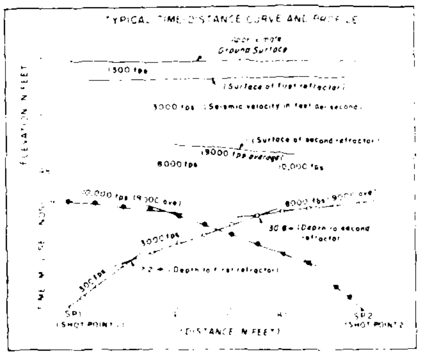


DIKE NO 2 LINE 8



SPILLWAY LINE 10





NOTES

1. SEE ~~SHEET 20~~ FOR APPROXIMATE LOCATIONS OF SEISMIC SURVEY LINES
2. SHOT POINT NUMBER 1 (SP1) FOR ANY LINE IS AT THE SOUTHERLY END AND SHOT POINT NUMBER 2 (SP2) AT THE NORTHERLY END
3. REFRACTIVE SEISMIC SURVEYS ACCOMPLISHED OCTOBER 1979 BY LOS ANGELES DISTRICT PERSONNEL WITH A 12 CHANNEL GEOMETRICS SEISMOGRAPH
4. INTERPRETATION OF DATA FROM THE REFRACTIVE SEISMIC SURVEYS WAS DONE BY LOS ANGELES DISTRICT PERSONNEL

MEAN SEA LEVEL

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY RLT

DRAWN BY

CHECKED BY

SUBMITTED BY

DATE APPROVED

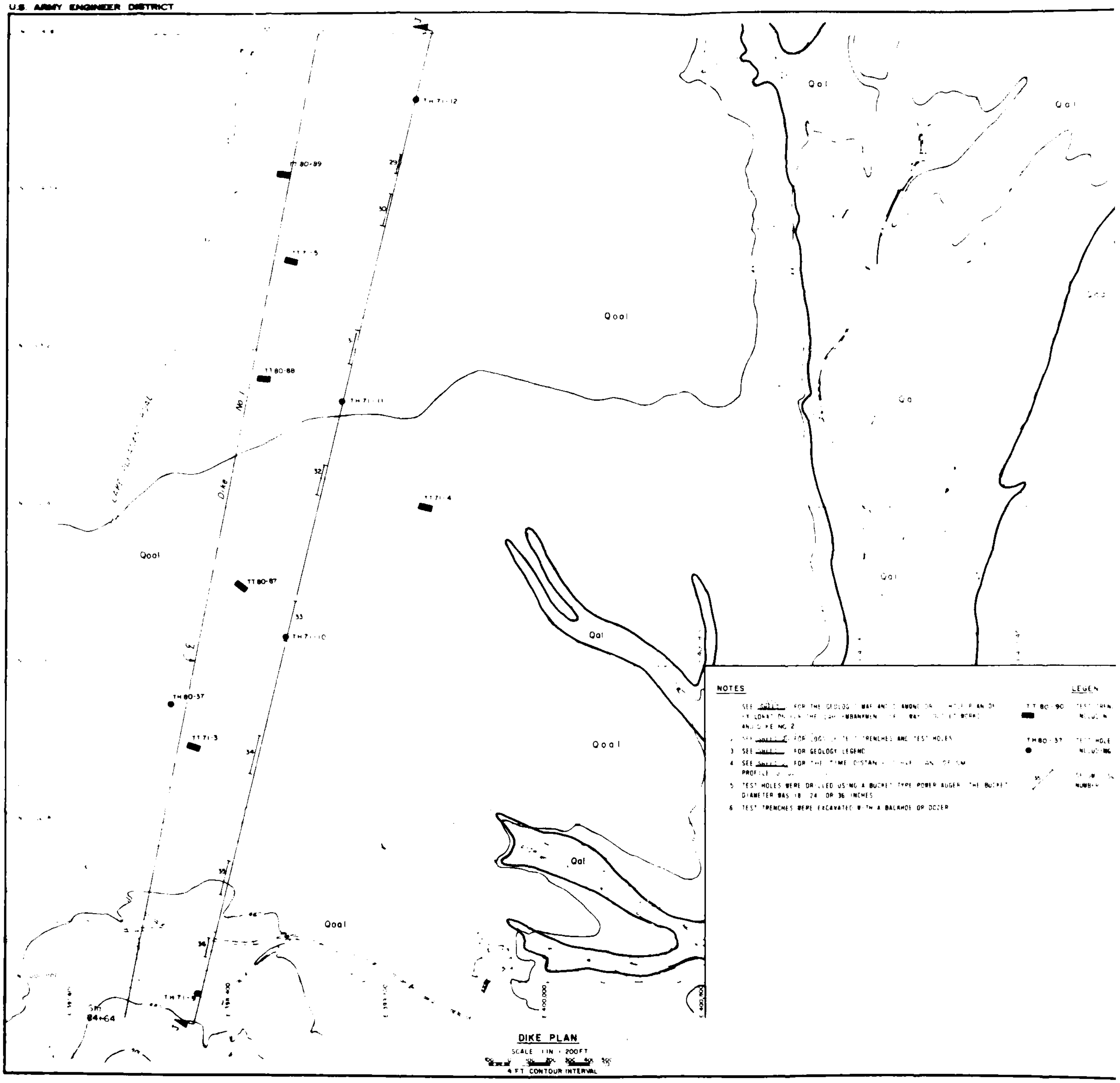
GILA RIVER BASIN
PHOENIX, ARIZONA AND VICINITY (INCLUDING NEW RIVER)

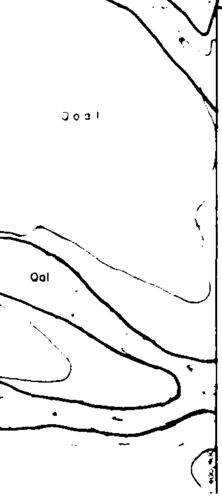
NEW RIVER DAM
SPILLWAY AND DIKE NO 2
REFRACTIVE SEISMIC SURVEYS
TIME-DISTANCE CURVES AND PROFILES

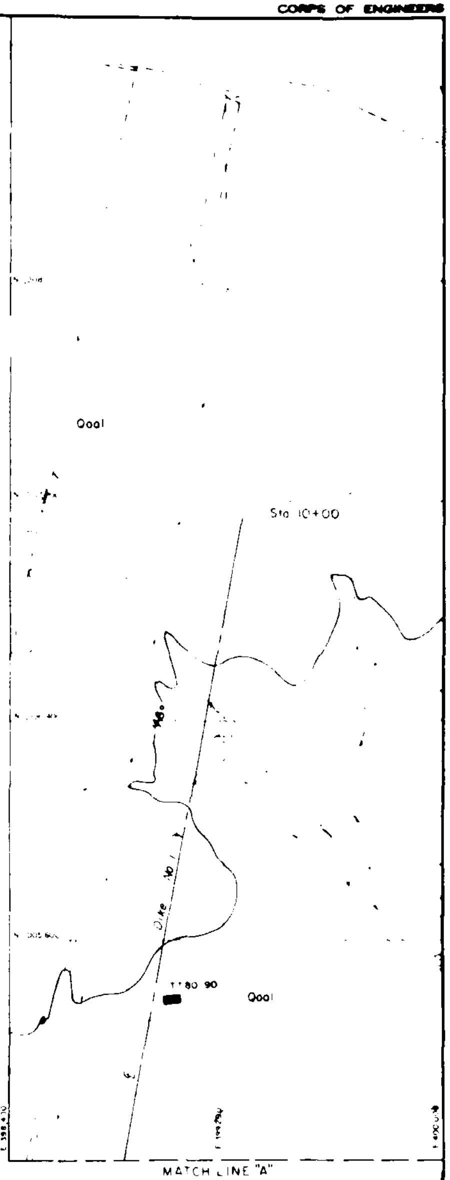
SPEC NO DACW09

DISTRICT FILE NO

SAFETY PAYS


U.S. ARMY ENGINEER DISTRICT
TH 71-12
TT 80-89
29
30
TT 71-5
Qoal
TT 80-88
TH 71-11
Dike No. 1
32
TT 71-4
Qoal
TT 80-87
33
TH 71-10
Qal
TH 80-37
TT 71-3
34
Qoal
35
Qoal
Qal
36
TH 71-9
Sta 84+64
Qal
Qal
Qa
Qal
NOTES
SEE SHEET FOR PROFILE OF TEST TRENCHES AND TEST HOLES
3 SEE SHEET FOR GEOLOGY LEGEND
4 SEE SHEET FOR THE TIME DISTAN
PROFILE
5 TEST HOLES WERE DRILLED USING A BUCKET TYPE POWER AUGER THE BUCKET DIAMETER WAS 18 24 OR 36 INCHES
6 TEST TRENCHES WERE EXCAVATED WITH A BACKHOE OR DOZER
LEGEN
TT 80-90
TH 80-37
NUMBER
DIKE PLAN
SCALE 1 IN = 200 FT
4 FT CONTOUR INTERVAL





NOTES

1. SEE SHEET FOR THE GEOLOGIC MAP AND DIAMOND DRILL HOLE PLAN OF EXPLORATION FOR THE DAM EMBANKMENT, SPILLWAY, OUTLET WORKS AND DIKE NO 2
2. SEE SHEET FOR LOGS OF TEST TRENCHES AND TEST HOLES
3. SEE SHEET FOR GEOLOGY LEGEND
4. SEE SHEET FOR THE TIME DISTANCE CURVES AND SEISMIC PROFILE
5. TEST HOLES WERE DRILLED USING A BUCKET TYPE POWER AUGER. THE BUCKET DIAMETER WAS 18, 24, OR 36 INCHES
6. TEST TRENCHES WERE EXCAVATED WITH A BACKHOE OR DOZER

LEGEND

TT 80-90	TEST TRENCH AND NUMBER INCLUDING YEAR EXCAVATED
TH 80-37	TEST HOLE AND NUMBER INCLUDING YEAR DRILLED
35	SEISMIC SURVEY LINE AND NUMBER

MEAN SEA LEVEL

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

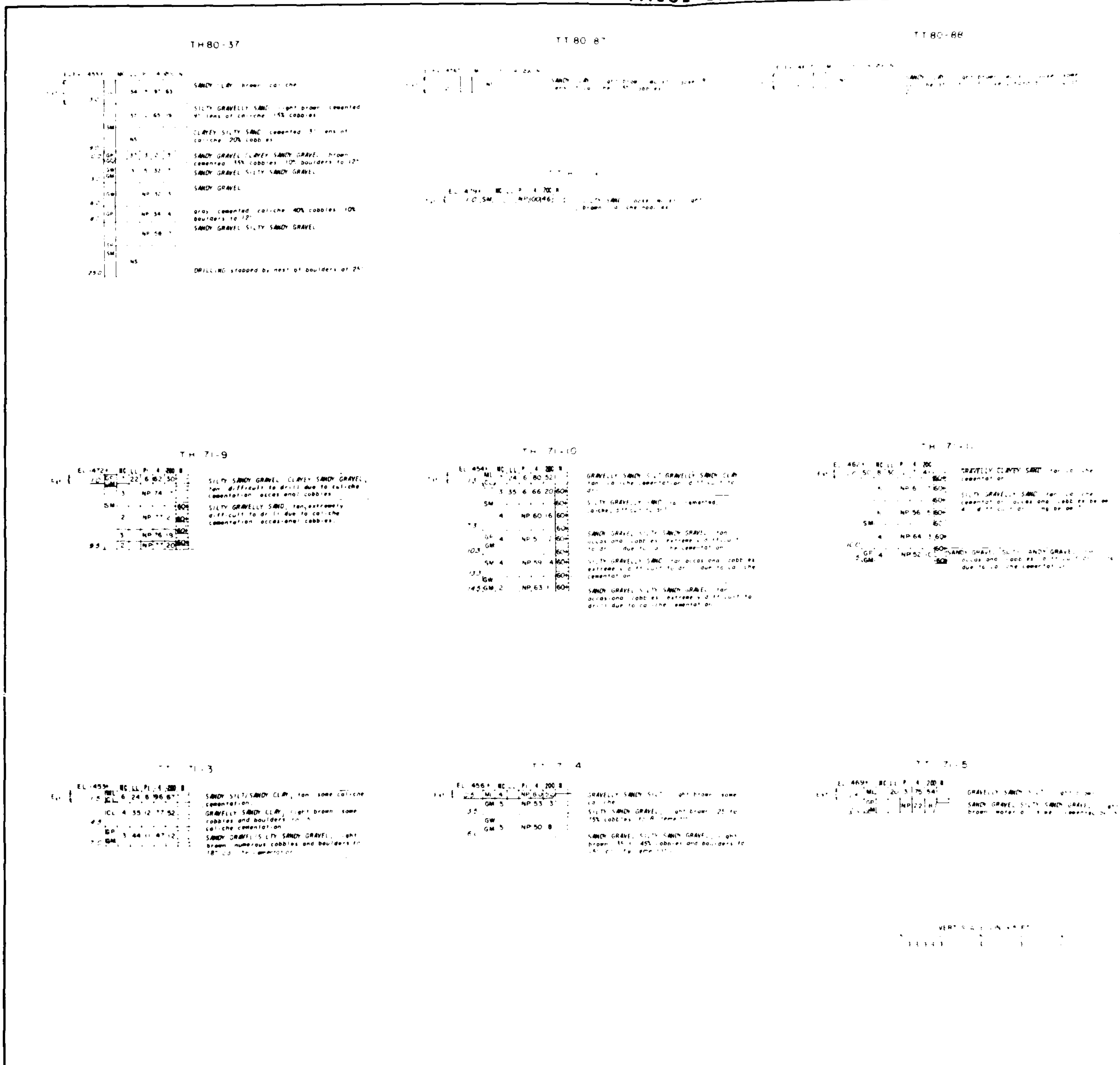
REVISIONS

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

GILA RIVER BASIN
PHOENIX, ARIZONA AND VICINITY (INCLUDING NEW RIVER)

NEW RIVER DAM
DIKE NO 1
GEOLOGY AND PLAN
OF EXPLORATION

DESIGNED BY: FLT
DRAWN BY: P.R.V.
CHECKED BY:
SUBMITTED BY:
DATE APPROVED:
SPEC NO DACW09-82-B-0016
DISTRICT FILE NO


TH 80-37
SANDY CLAY, brown, caliche
SILTY GRAVELLY SAND, light brown, cemented
CLAYEY SILTY SAND, cemented
SANDY GRAVEL
DRILLING stopped by nest of boulders at 25'
TH 71-9
SILTY GRAVELLY SAND, tan, extremely difficult to drill due to caliche cementation, occasional cobbles
TH 71-3
SANDY SILT/SANDY CLAY, tan, some caliche cementation
GRAVELLY SANDY CLAY, light brown, some cobbles and boulders
SANDY GRAVEL, SILTY SANDY GRAVEL

VALUE ENGINEERING PAYS

TT 80-88

TT 80-89

TH 71-11

GRAVELLY CLAYEY SAND

SILTY GRAVELLY SAND

SANDY GRAVEL

TH 71-12

SANDY CLAY, tan, caliche cementation

GRAVELLY CLAYEY SAND, tan, caliche cementation, difficult to drill.

CLAYEY GRAVELLY SAND.

SILTY GRAVELLY SAND, tan, caliche cementation, occasional cobbles and boulders to 3', extremely difficult to drill.

● Penetrometer would not seat

NOTES:

SEE SHEET 26 FOR NOTES, LEGEND, AND BASIS FOR CLASSIFICATIONS

GRAVELLY SANDY SILT

SANDY GRAVEL, SILTY SANDY GRAVEL

DATUM IS MEAN SEA LEVEL

REVISIONS

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

GILA RIVER BASIN

NEW RIVER DAM

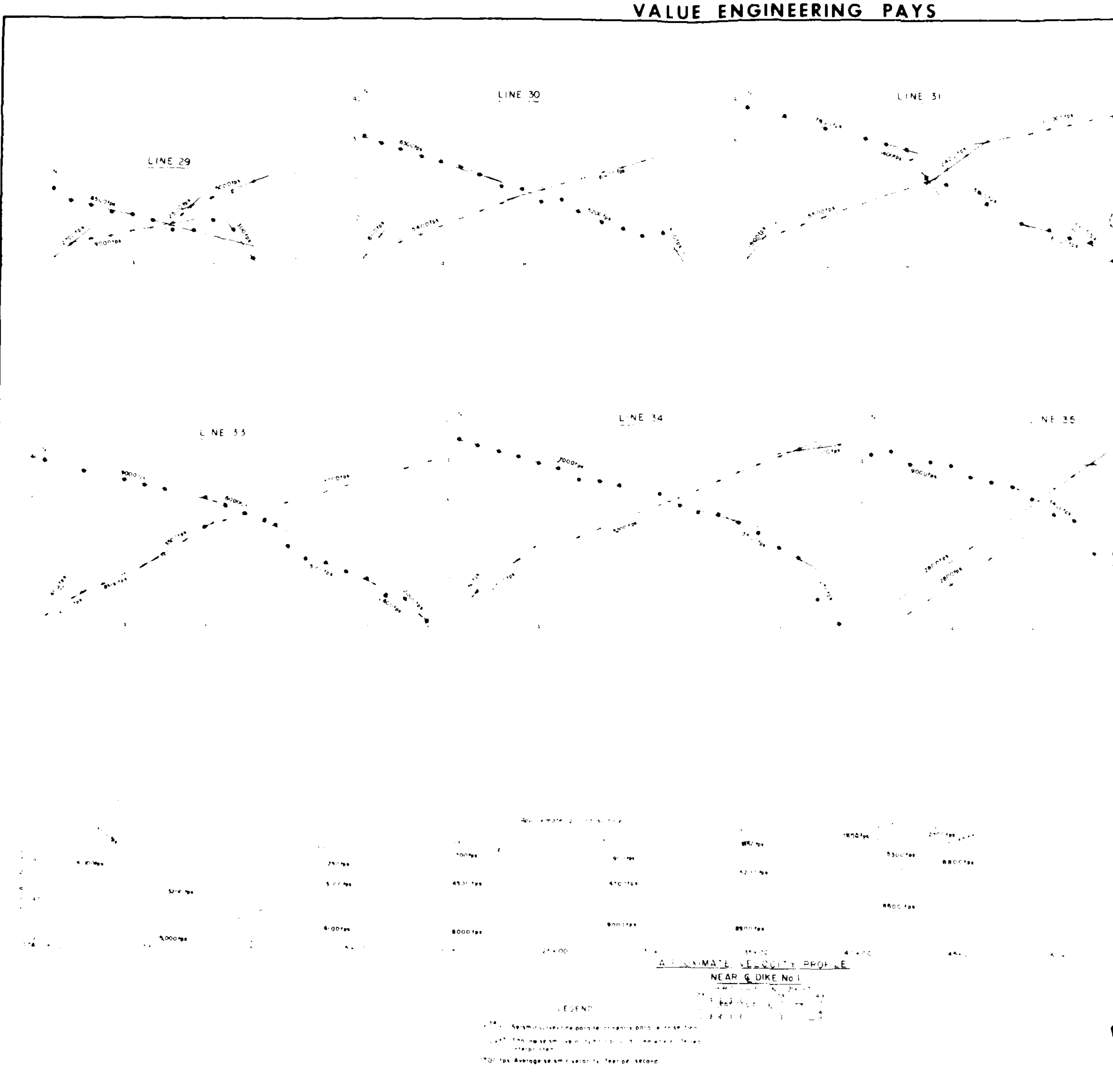
DIKE NO 1 FOUNDATION
SOIL LOGS

DRAWN BY J F B

CHECKED BY ATR

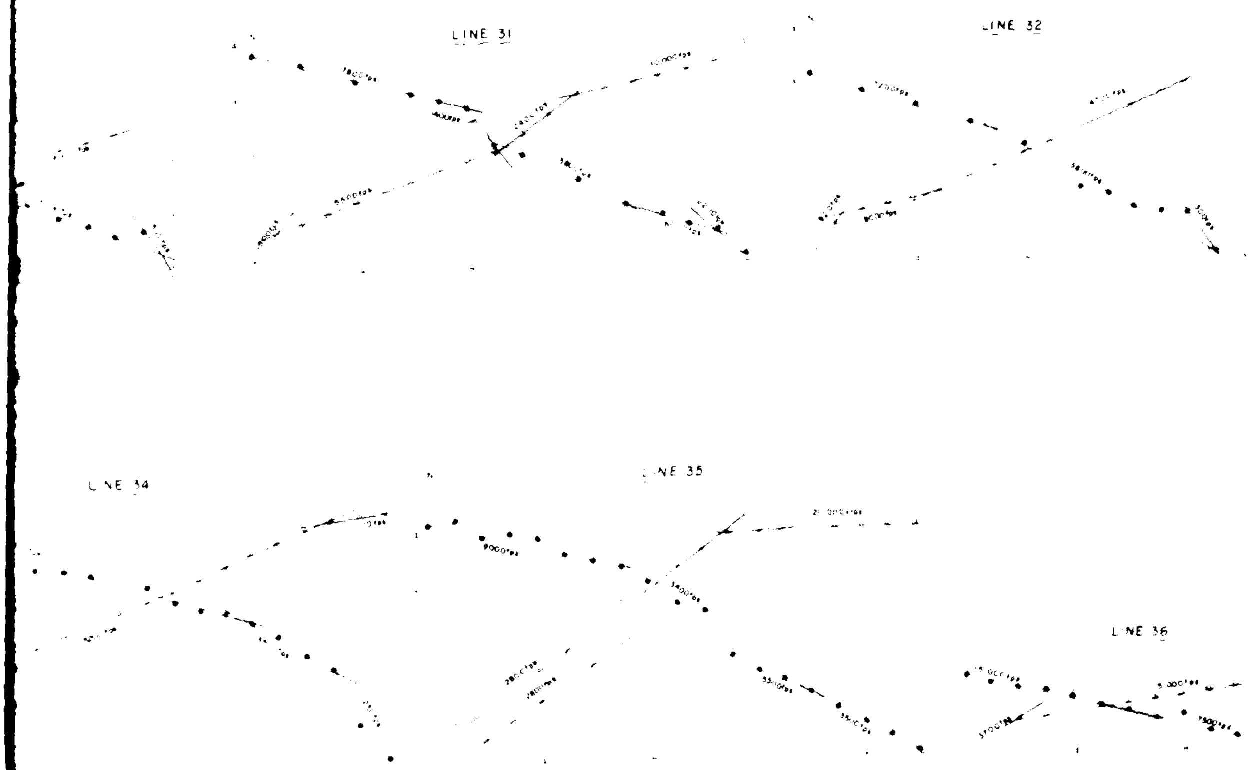
SAFETY PAYS

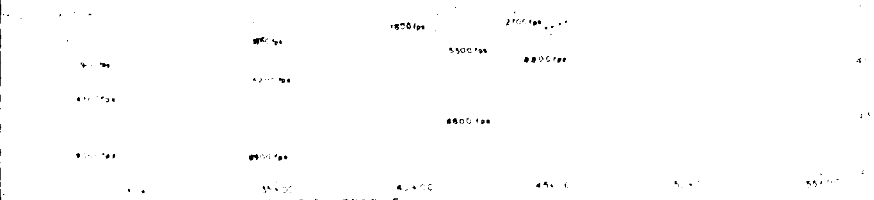
VALUE ENGINEERING PAYS



SAFETY PAYS

VALUE ENGINEERING PAYS





APPROXIMATE VELOCITY PROFILE
NEAR ℄ DIKE No I

NOTES

THE REFRACTIVE SEISMIC SURVEYS WERE CONDUCTED ALONG THE ORIGINAL ALIGNMENT OF THE DIKE.

THE INTERPRETATION OF DATA FROM THE REFRACTIVE SEISMIC SURVEYS WAS DONE BY ARIZONA TESTING LABORATORIES.

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

U S ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

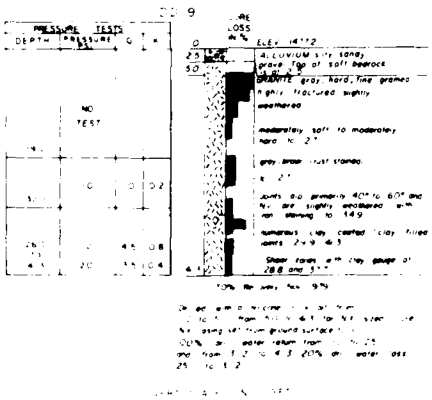
DESIGNED BY R.
DRAWN BY
CHECKED BY

NEW RIVER DAM
DIKE No I
REFRACTIVE SEISMIC SURVEYS
TIME-DISTANCE CURVES AND PROFILE

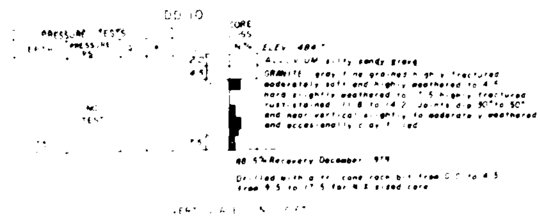
SUBMITTED BY
DATE
APPROVED
SPEC NO DACW09
DISTRICT FILE NO
SHEET 22 OF 81

SAFETY PAYS

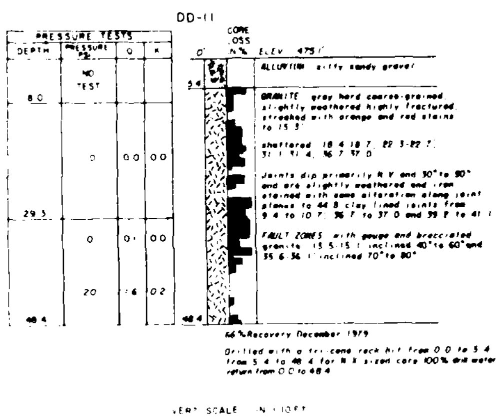
DIKE NO 2

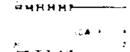


DIKE NO 2



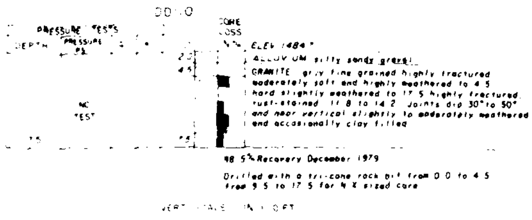
DIKE NO 2

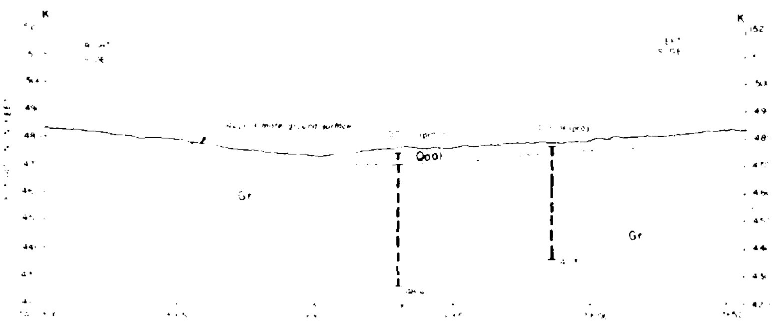




VALUE ENGINEERING PAYS

DIKE NO 2





PROFILE ALONG ℄ DIKE NO 2

NOTES

1. SEE SHEET 3 FOR THE GEOLOGIC MAP AND DIAMOND DRILL HOLE PLAN OF EXPLORATION
2. SEE SHEET 3 FOR LOCATION OF DIKE NO 2 PROFILE
3. SEE SHEET 5 FOR LEGEND
4. SEE SHEETS 3 AND 4 FOR ADDITIONAL LEGEND
5. DIAMOND DRILL HOLE ELEVATIONS ARE SURVEYED
6. ALL SOIL CLASSIFICATIONS SHOWN ON THIS SHEET ARE VISUAL

DATUM IS MEAN SEA LEVEL

REVISIONS

U S ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

GILA RIVER BASIN
PHOENIX ARIZONA AND VICINITY INCLUDING NEW RIVER

NEW RIVER DAM
DIKE NO 2
PROFILE AND GEOLOGIC LOGS OF
DIAMOND DRILL HOLES

SUBMITTED BY
APPROVED

SAFETY PAYS

LEGEND

QUATERNARY:

Qoal **OLDER ALLUVIUM**: Flood plain, alluvial fan and valley fill deposits, residual soil, colluvium and slopewash; clay, silt, sand, gravel, cobbles and occasional boulders; poorly to well consolidated, locally caliche cemented.

TERTIARY:

Tvt$_1$ **VOLCANIC ASH-FALL TUFF**: light pink to salmon pink, moderately soft to moderately hard, moderately to highly weathered; well sorted fine to coarse grained ash consisting of glass, pumice and volcanic rock fragments, crudely to well stratified.

Tvt$_2$ **VOLCANIC LAPILLI ASH-FLOW TUFF**: mottled salmon-pink to gray, moderately hard, slightly to moderately weathered; unsorted and non-stratified coarse ash and lapilli size fragments of glass, volcanic rock and pumice in an ash matrix.

Tvt$_3$ **VOLCANIC ASH-FALL TUFF**: light purple, moderately hard to hard, generally unweathered, slightly to moderately welded; crudely stratified, fine to medium grained ash consisting mostly of pumice and glass fragments.

Tvt$_4$ **VOLCANIC ASH-FALL WELDED TUFF**: light purple to purplish-red, hard, generally unweathered, moderately welded, with bands of fine to coarse grained ash consisting mostly of glass and pumice fragments.

Tvta **TUFFACEOUS AGGLOMERATE**: mottled grey to pinkish red, moderately soft to moderately hard, moderately to highly weathered; unsorted, nonstratified, angular volcanic fragments in a vesicular, glassy matrix.

Tvfb **VOLCANIC FLOW BRECCIA**: reddish-brown, crumbly to locally hard, massive, moderately to highly weathered; angular to subrounded vesicular andesitic to basaltic scoria blocks and lapilli fragments in a cindery matrix; some secondary mineralization.

Tva **VOLCANIC ANDESITE**: medium to dark gray, hard, generally unweathered, aphanitic, occasional flow banding, typically highly fractured, platy to blocky; andesite near flow breccia contacts is brownish-purple to mottled purple-maroon-gray, slightly to moderately weathered, locally brecciated and rehealed.

Tvpa **VOLCANIC PORPHYRITIC ANDESITE**: pinkish-
and bands, hard, slightly weathered, typica
ly blocky, locally brecciated and reheale
pinkish-gray, platy, moderately hard and

PRECAMBRIAN:

in **INTRUSIVE ROCK**: includes pods, lenses an
to reddish-black aphanitic granitic rock
weathered, locally brecciated and reheale

gr **GRANITE**. light-gray to reddish-brown, me
hard to hard, typically highly fractured
weathered, locally sheared and altered.

qd **QUARTZ DIORITE**: medium to dark gray,
to hard, typically highly fractured, sl
sheared and altered.

d **DIORITE**: medium to whitish gray, med
to hard, typically highly fractured, varia
altered.

Strike and dip (in degrees) of vertical

70 Strike and dip (in degrees) of individua

N 75 W
85 N Strike and dip of joint or layering in s

1"-4" Infilled open fracture (widths shown) in

: pinkish-grey with reddish-brown lenses
ered, typically highly fractured, occasional-
and rehealed; upper part of flow is light
, hard and moderately weathered.

s, lenses and dikes of a medium-gray to black
anitic rock; moderately soft to hard, variably
and rehealed with clay gouge.

sh-brown, medium to coarse grained, moderately
fractured and blocky, slightly to moderately
altered.

ik gray, fine to medium grained, moderately hard
tured, slightly to moderately weathered; locally

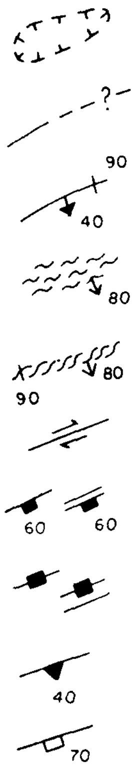
medium to coarse grained, moderately soft
red, variably weathered; locally sheared and

of vertical dike.

individual shear.

yering in spillway wall.

shown) in spillway wall.



Scour channel incised in bedrock.

Contact between geologic units, dashed where approximate, queried where uncertain.

Contact, showing dips.

Shear zones, showing average dips

Shear zones, showing average dips.

Fault (?), showing relative horizontal movement.

Strike and dip (in degrees) of joint or joint set.

Strike of vertical joint or joint set.

Srike and dip (in degrees) of layering.

Strike and dip (in degrees) of dike.

h	hard
mh	moderately hard
ms	moderately soft
s	soft
uw	unweathered
sw	slightly weathered
mw	moderately weathered
hw	highly weathered
sf	slightly fractured
mf	moderately fractured
hf	highly fractured
sh	shattered
br	brecciated, breccia
bl	blocky
pl	platy
sz	shear zone
ij	indistinct joints
rs	rust stained
ss	slickensides
shrs	shears
cg	clay gouge
cc	calcium carbonate
dg	decomposed granite
y/b	yellow-brown
jnt(s)	joint(s)
ind	indistinct
m	massive
r/b	red-brown
c	clay

GENERAL NOTES

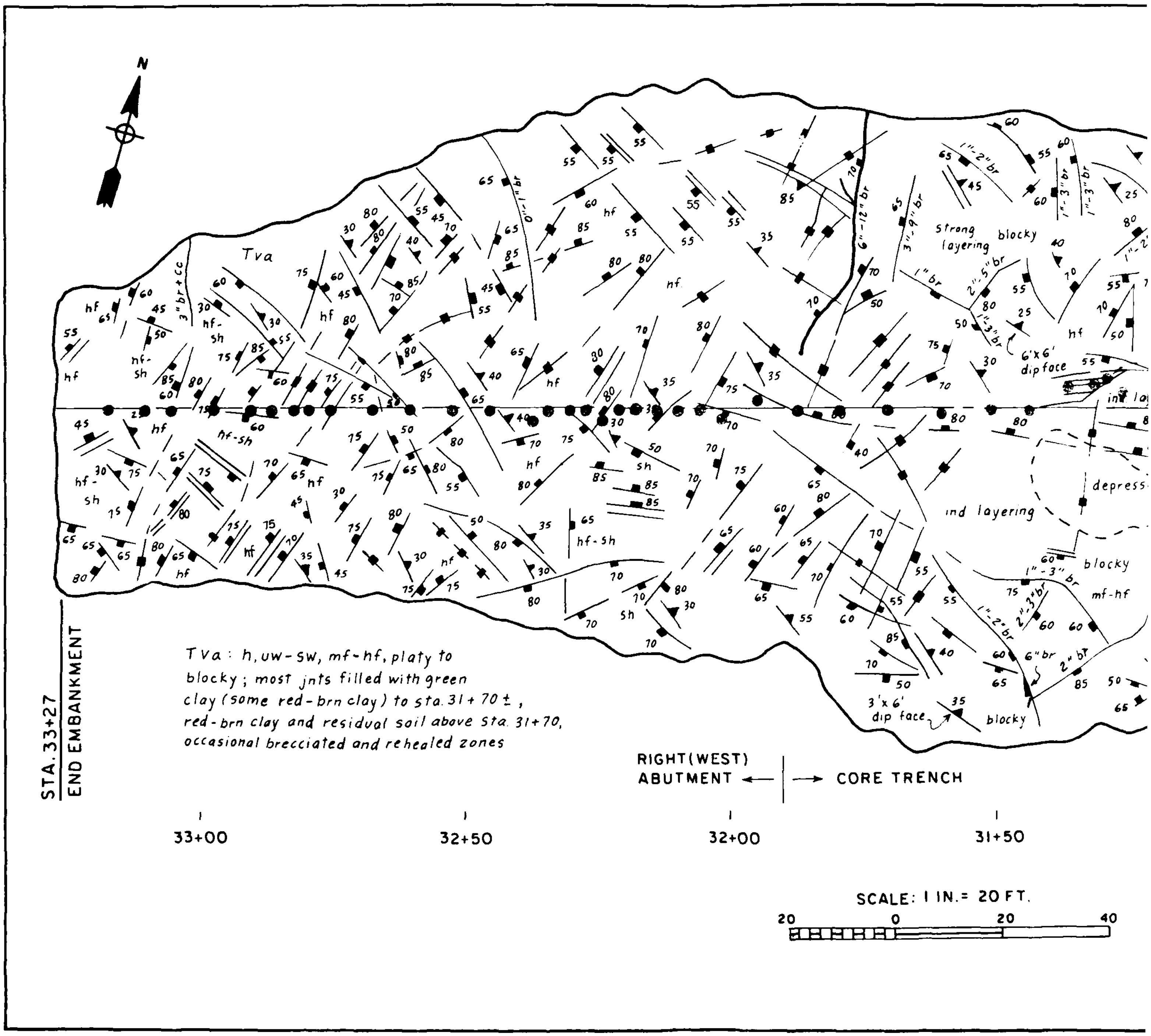
1. See Plates 29 through 33 and 37 through 44 for geologic maps of dam foundation, outlet works, spillway, and dike no.1 .
2. See Plate 6 for additional legend.
3. Soil classifications shown on geologic maps are visual.
4. Rock classifications shown on geologic maps are generally based on petrographic analyses.

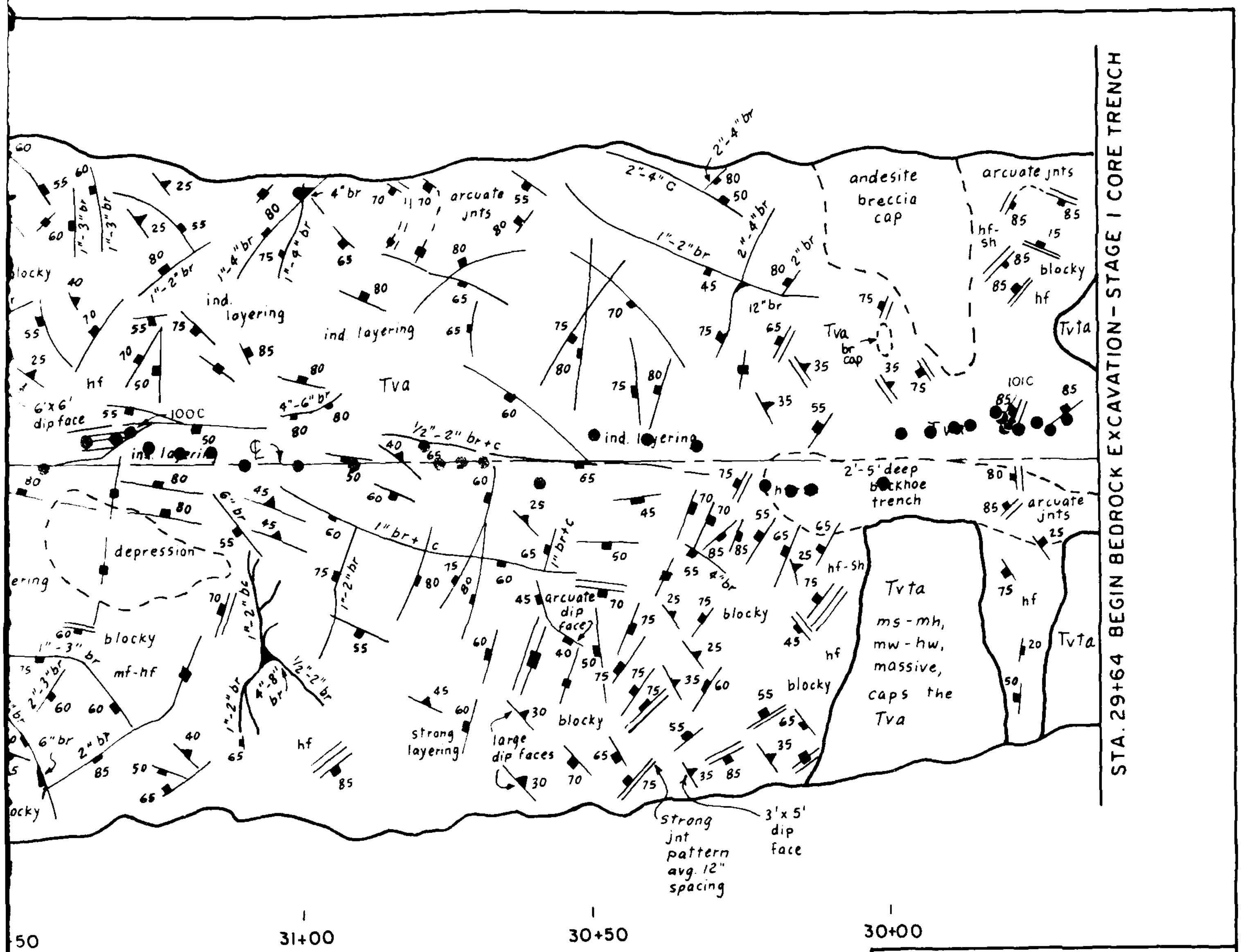
GILA RIVER BASIN
PHOENIX, AZ. AND VICINITY (INCL. NEW RIVER)

NEW RIVER DAM

LEGEND AND GENERAL NOTES

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES CORPS OF ENGINEERS


N
Tva
Tva : h, uw-sw, mf-hf, platy to blocky ; most jnts filled with green clay (some red-brn clay) to sta. 31+70 ±, red-brn clay and residual soil above Sta. 31+70, occasional brecciated and rehealed zones
STA. 33+27
END EMBANKMENT
RIGHT (WEST) ABUTMENT
CORE TRENCH
33+00
32+50
32+00
31+50
SCALE: 1 IN. = 20 FT.
20
0
20
40
Strong layering
blocky
6'x6' dip face
3'x6' dip face
ind layering
depress
mf-hf
hf-sh



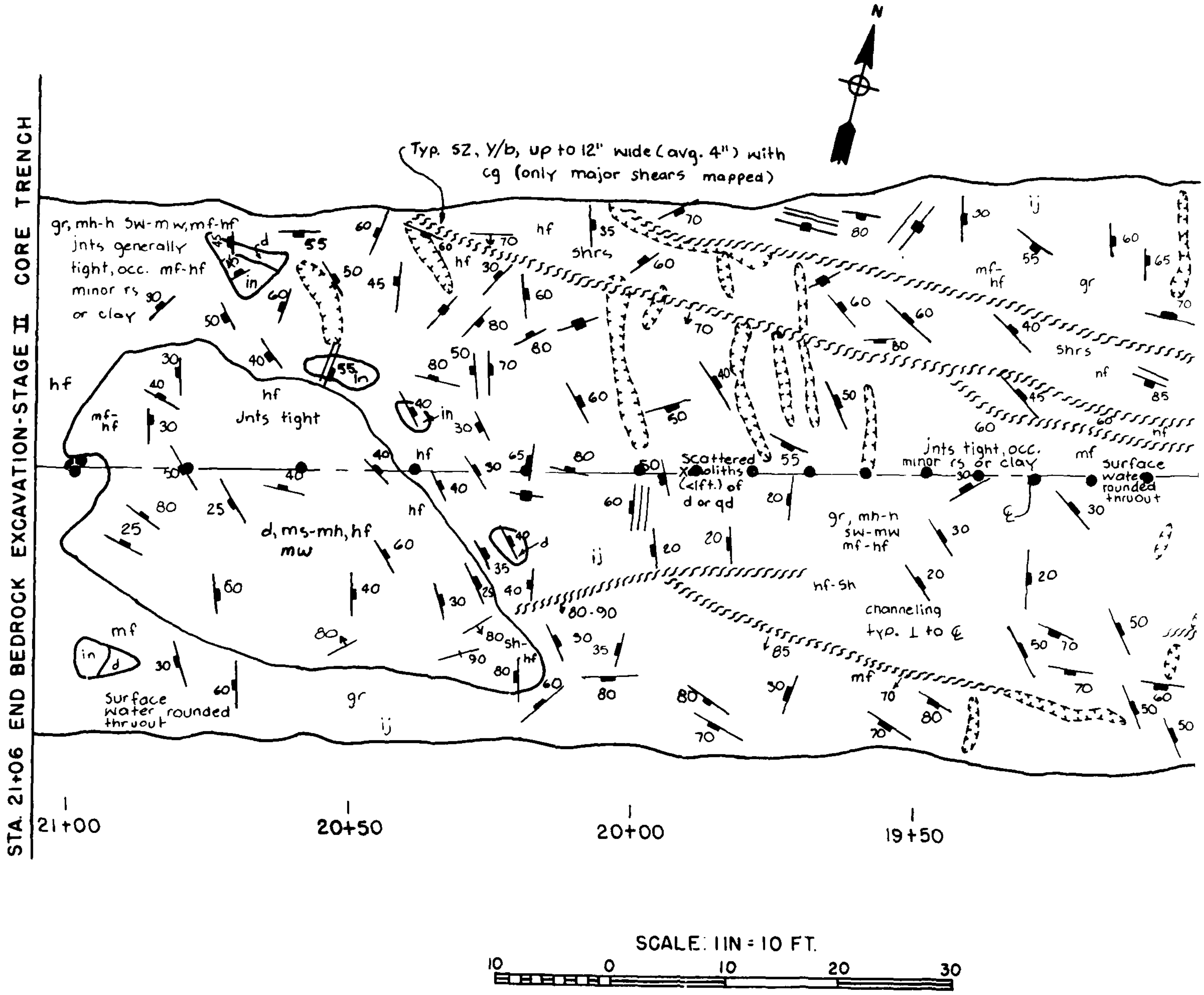
FT.
20 40

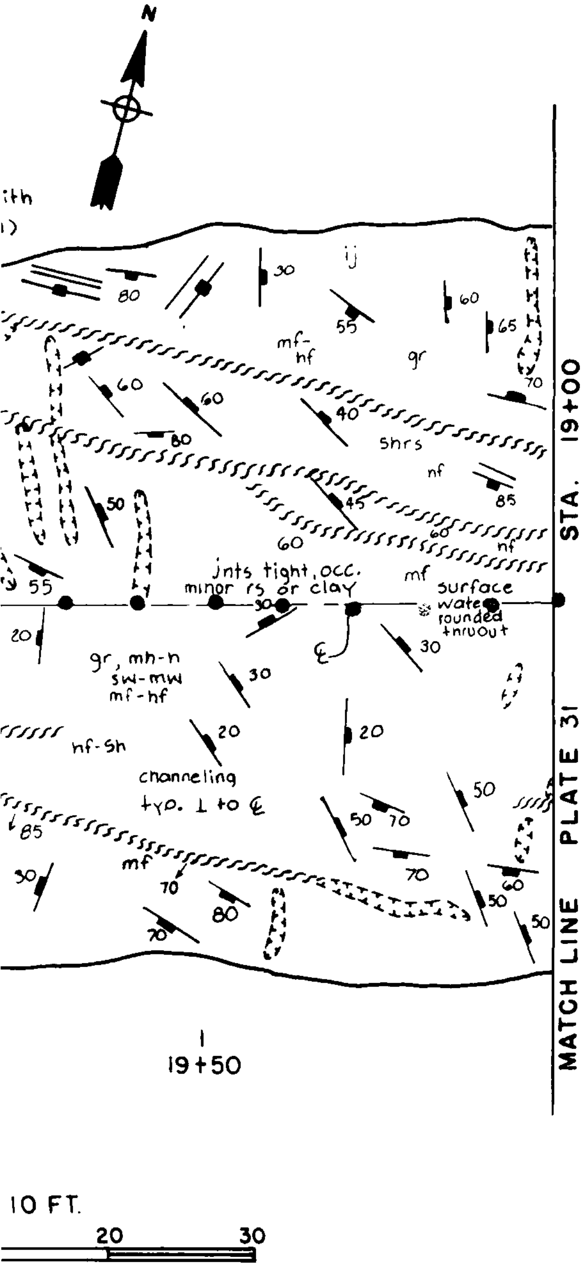
NOTES:

1. SEE PLATE 28 FOR LEGEND AND GENERAL NOTES.
2. SEE PLATE 33 FOR ADDITIONAL LEGEND.

GILA RIVER BASIN
PHOENIX AZ. AND VICINITY (INCL. NEW RIVER)
NEW RIVER DAM
CORE TRENCH AND RIGHT (WEST) ABUTMENT
FOUNDATION GEOLOGY
STA. 33+27 TO STA. 29+64

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES CORPS OF ENGINEERS


STA 21+06 END BEDROCK EXCAVATION-STAGE II CORE TRENCH
N
Typ. SZ, Y/b, up to 12" wide (avg. 4") with cg (only major shears mapped)
gr, mh-h sw-mw, mf-hf jnts generally tight, occ. mf-hf minor rs or clay
hf
mf-hf
hf
Jnts tight
d, ms-mh, hf mw
in
d
shrs
Scattered Xenoliths (<1ft.) of d or qd
jnts tight, occ. minor rs or clay
surface water rounded thruout
gr, mh-h sw-mw mf-hf
hf-sh
channeling typ. ⊥ to ℄
mf
Surface water rounded thruout
gr
ij
80-90
21+00
20+50
20+00
19+50
SCALE: 1 IN = 10 FT.
10
0
10
20
30



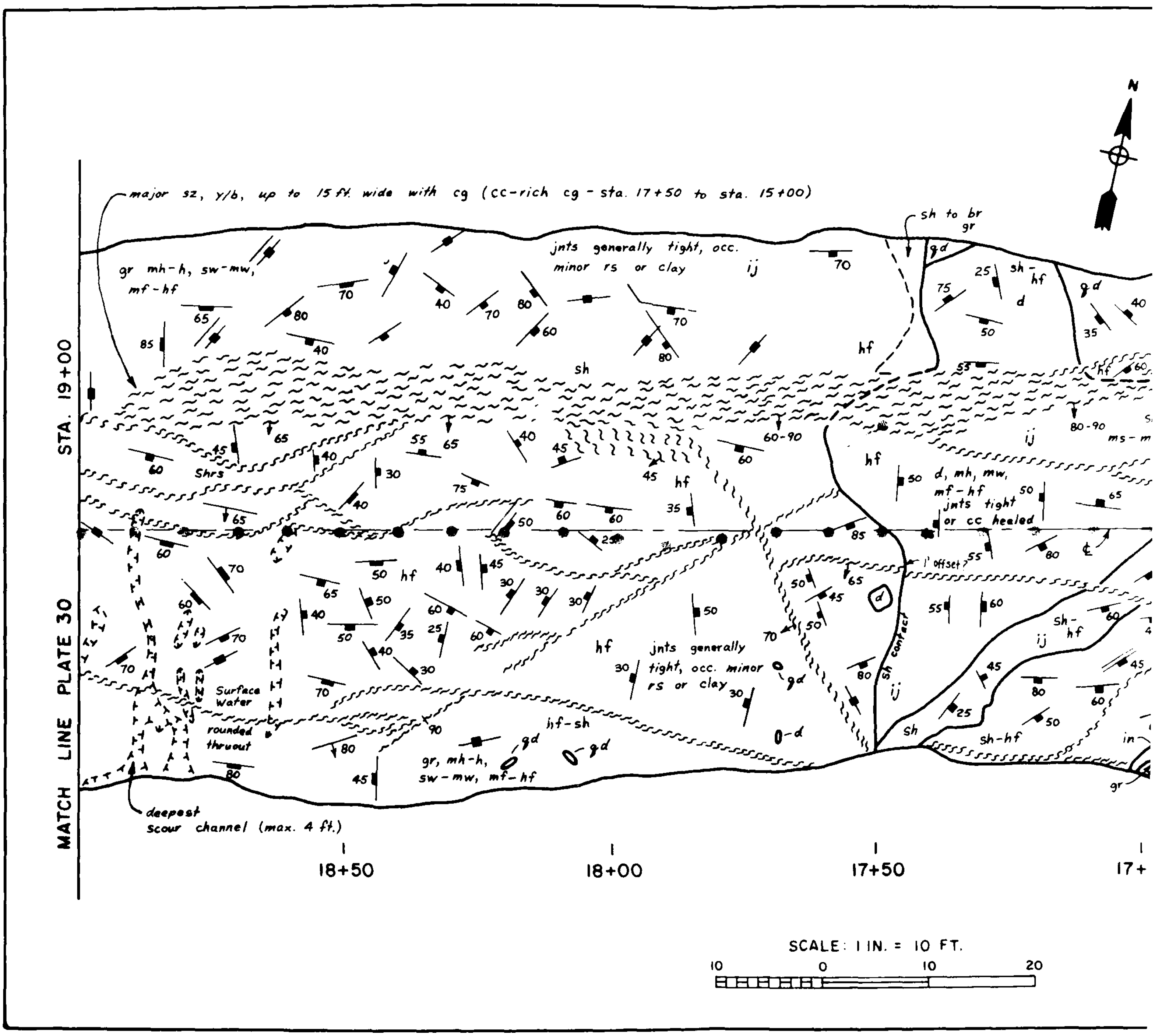
NOTES:

1. SEE PLATE 28 FOR LEGEND AND GENERAL NOTES
2. SEE PLATE 33 FOR ADDITIONAL LEGEND.

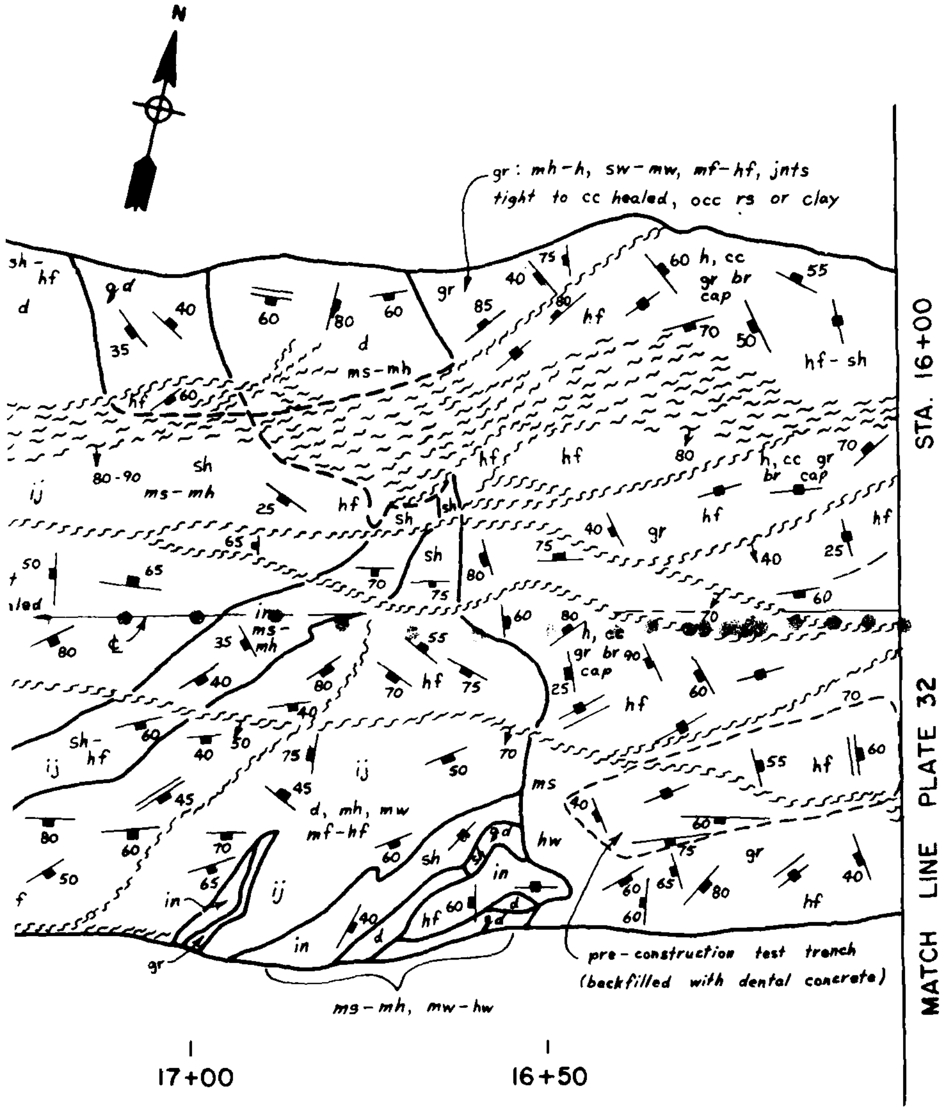
GILA RIVER BASIN
PHOENIX AZ. AND VICINITY (INCL. NEW RIVER)

NEW RIVER DAM
CORE TRENCH
FOUNDATION GEOLOGY
STA. 21+06 TO STA. 19+00

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES CORPS OF ENGINEERS



major sz, y/b, up to 15 ft. wide with cg (cc-rich cg - sta. 17+50 to sta. 15+00)
sh to br gr
N
gr mh-h, sw-mw, mf-hf
jnts generally tight, occ. minor rs or clay
ij
gd
sh-hf
d
hf
sh
Shrs
d, mh, mw, mf-hf jnts tight or cc healed
1' offset ?
sh contact
Surface water
rounded thruout
hf-sh
gr, mh-h, sw-mw, mf-hf
jnts generally tight, occ. minor rs or clay
sh-hf
deepest scour channel (max. 4 ft.)
STA. 19+00
MATCH LINE PLATE 30
18+50
18+00
17+50
SCALE: 1 IN. = 10 FT.
10
0
10
20



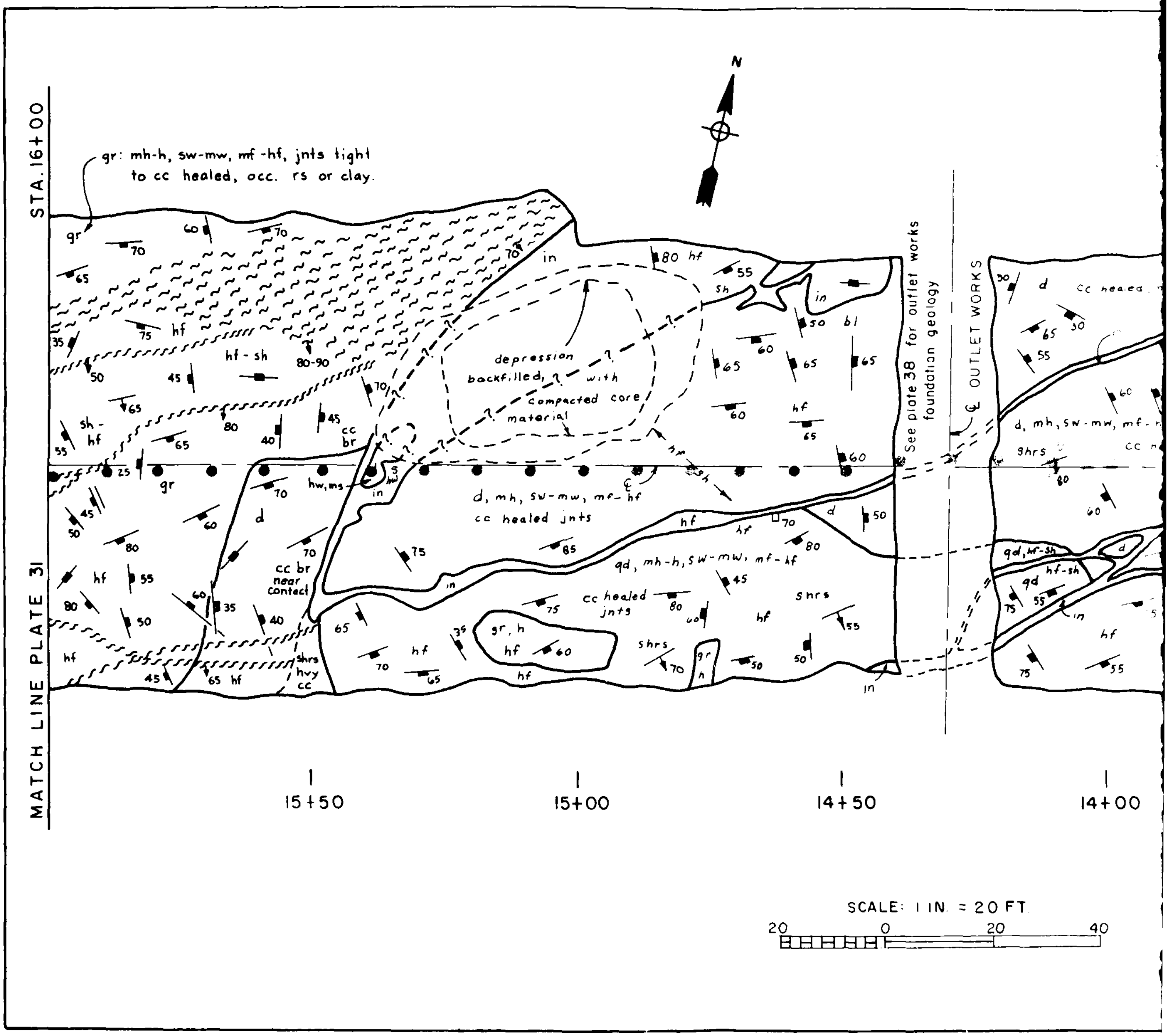
NOTES:

1. SEE PLATE 28 FOR LEGEND AND GENERAL NOTES.
2. SEE PLATE 33 FOR ADDITIONAL LEGEND.

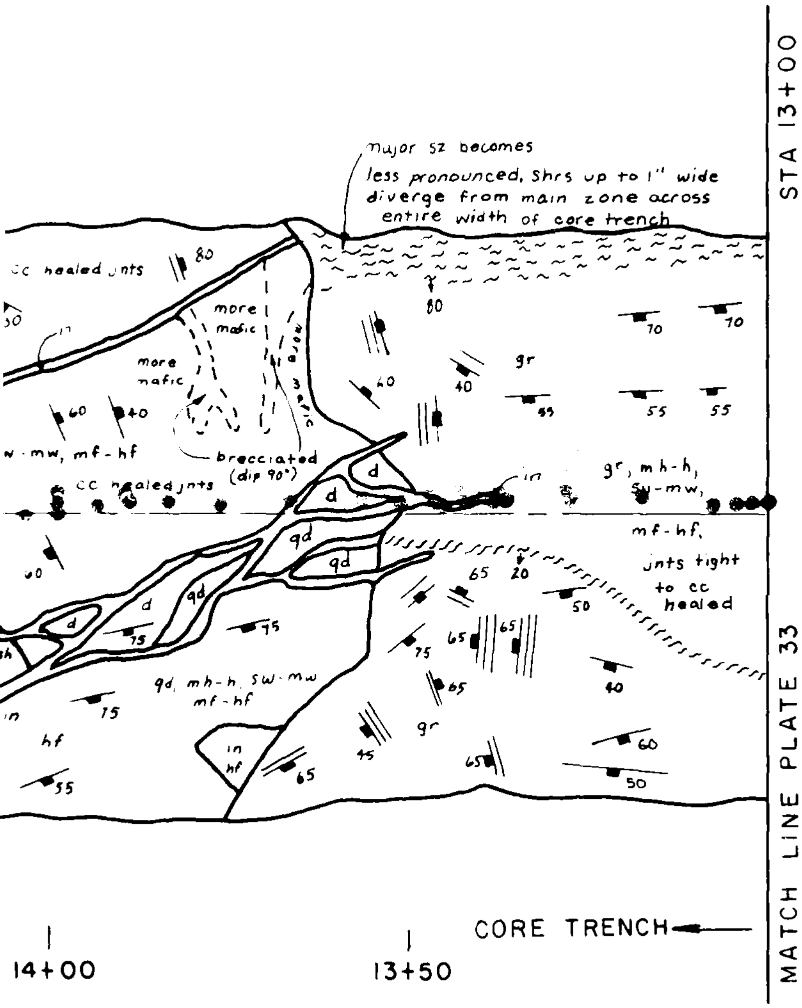
GILA RIVER BASIN
PHOENIX AZ. AND VICINITY (INCL. NEW RIVER)

NEW RIVER DAM
CORE TRENCH
FOUNDATION GEOLOGY
STA. 19+00 TO STA. 16+00

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES CORPS OF ENGINEERS



N
STA. 16+00
MATCH LINE PLATE 31
gr: mh-h, sw-mw, mf-hf, jnts tight to cc healed, occ. rs or clay.
hf-sh
sh-hf
cc br
hw, ms
depression backfilled, with compacted core material
d, mh, sw-mw, mf-hf
cc healed jnts
qd, mh-h, sw-mw, mf-hf
cc healed jnts
cc br near contact
shrs hvy cc
gr, h
shrs
See plate 38 for outlet works foundation geology
OUTLET WORKS
d, mh, sw-mw, mf-
qd, hf-sh
15+50
15+00
14+50
14+00
SCALE: 1 IN. = 20 FT.
20
0
20
40



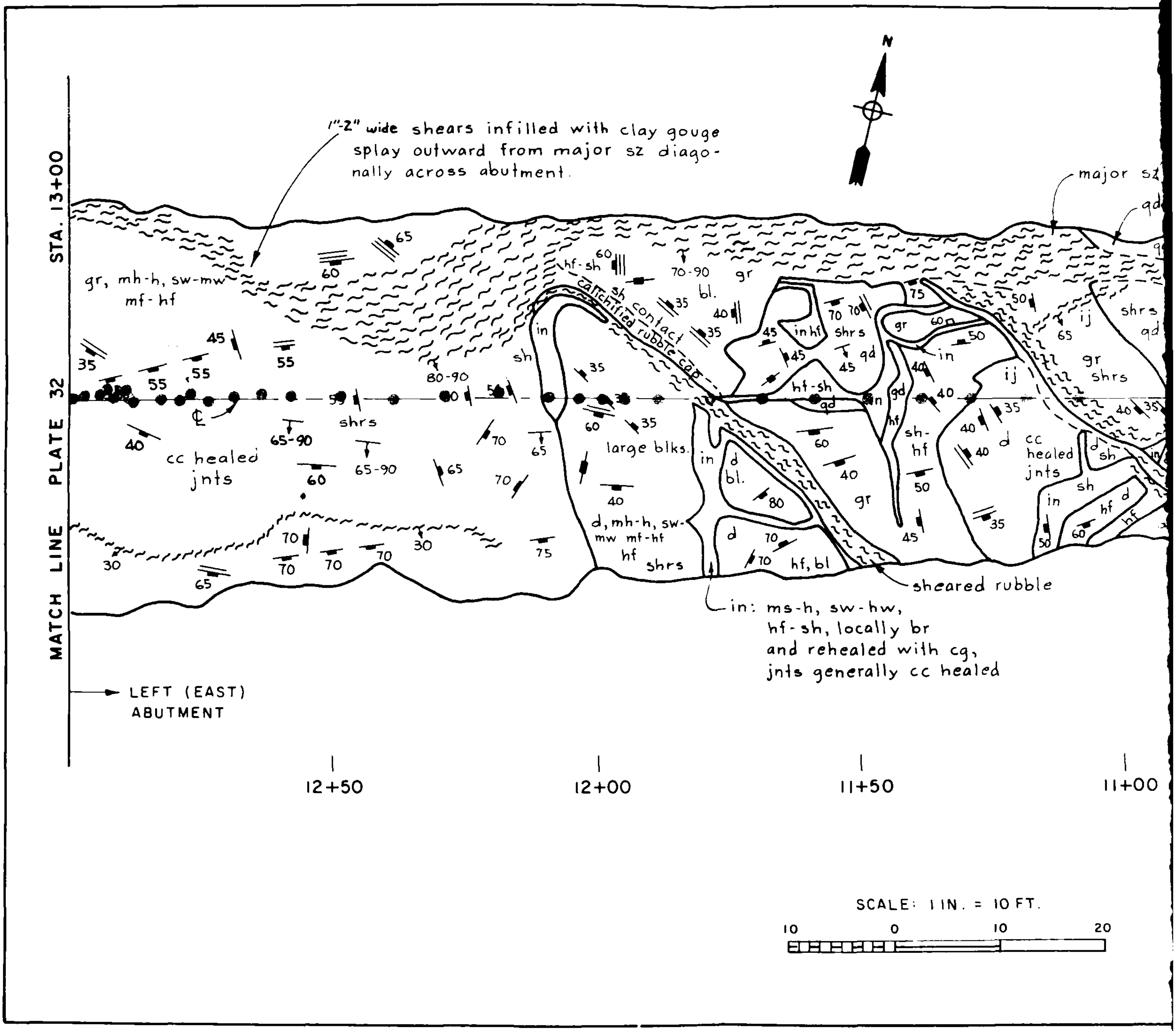
NOTES:

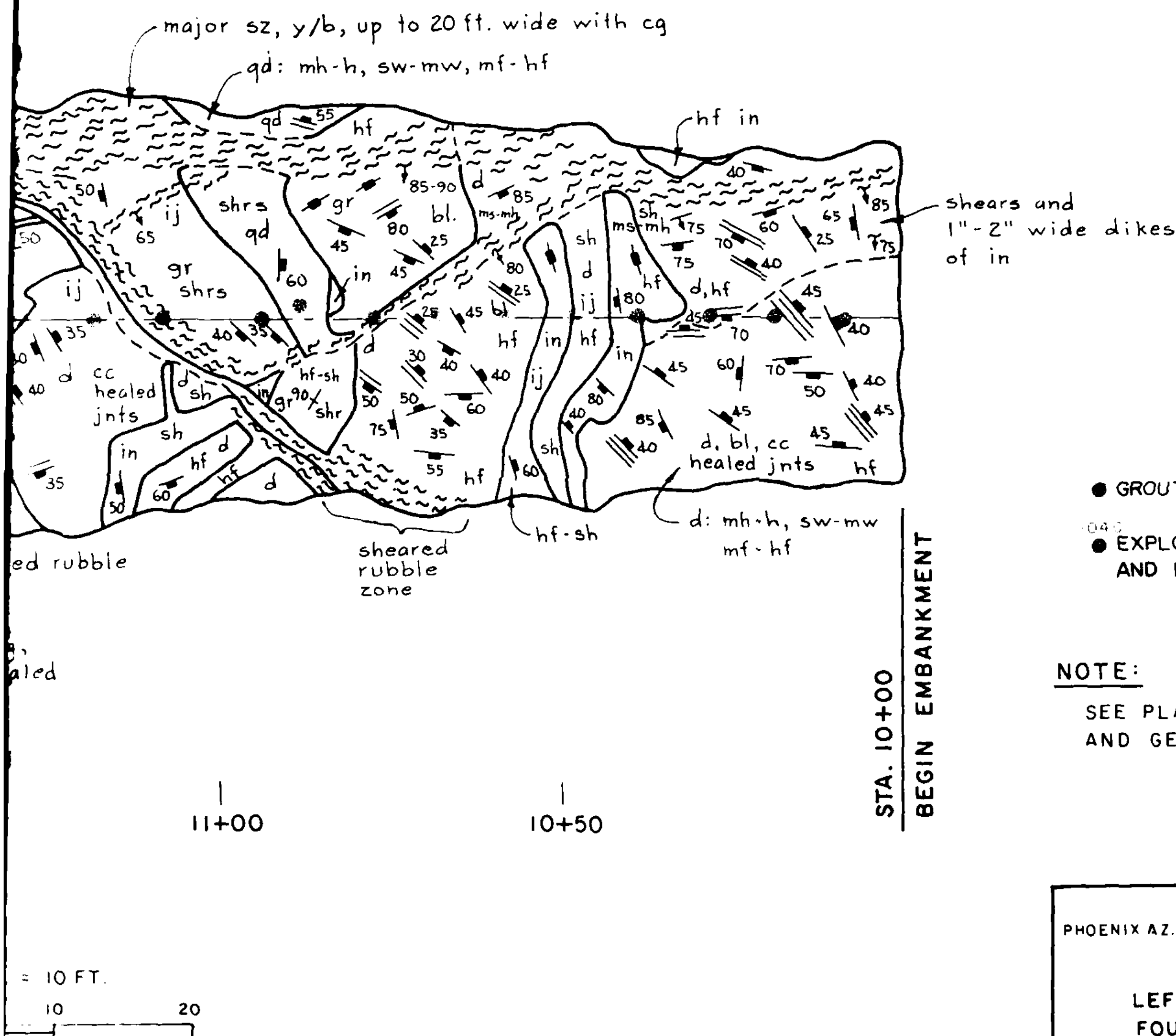
1. SEE PLATE 28 FOR LEGEND AND GENERAL NOTES.
2. SEE PLATE 33 FOR ADDITIONAL LEGEND.

GILA RIVER BASIN
PHOENIX AZ. AND VICINITY (INCL. NEW RIVER)

NEW RIVER DAM
CORE TRENCH
FOUNDATION GEOLOGY
STA. 16+00 TO STA. 13+00

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES CORPS OF ENGINEERS


MATCH LINE PLATE 32 STA. 13+00
1"-2" wide shears infilled with clay gouge splay outward from major sz diagonally across abutment.
major sz
gr, mh-h, sw-mw mf-hf
cc healed jnts
shrs
large blks.
d, mh-h, sw-mw mf-hf hf shrs
hf-sh
sh contact
calcitified rubble cap
sheared rubble
in: ms-h, sw-hw, hf-sh, locally br and rehealed with cg, jnts generally cc healed
LEFT (EAST) ABUTMENT
12+50
12+00
11+50
11+00
SCALE: 1 IN. = 10 FT.
10
0
10
20



LEGEND

● GROUT HOLE

● EXPLORATORY GROUT HOLE AND NUMBER

NOTE:

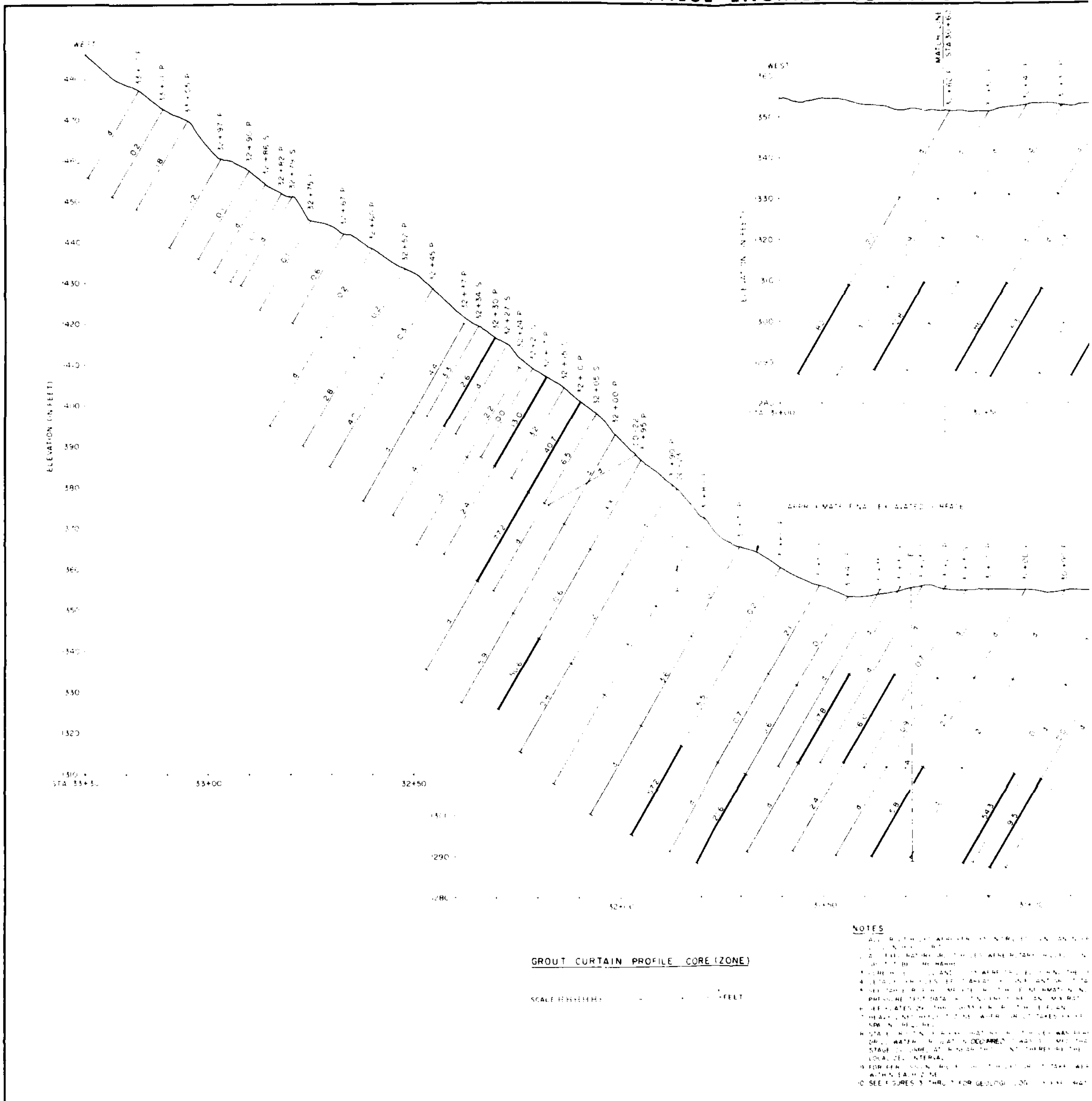
SEE PLATE 28 FOR LEGEND AND GENERAL NOTES.

GILA RIVER BASIN
PHOENIX AZ. AND VICINITY (INCL. NEW RIVER)

NEW RIVER DAM
LEFT (EAST) ABUTMENT
FOUNDATION GEOLOGY
STA. 13+00 TO STA. 10+00

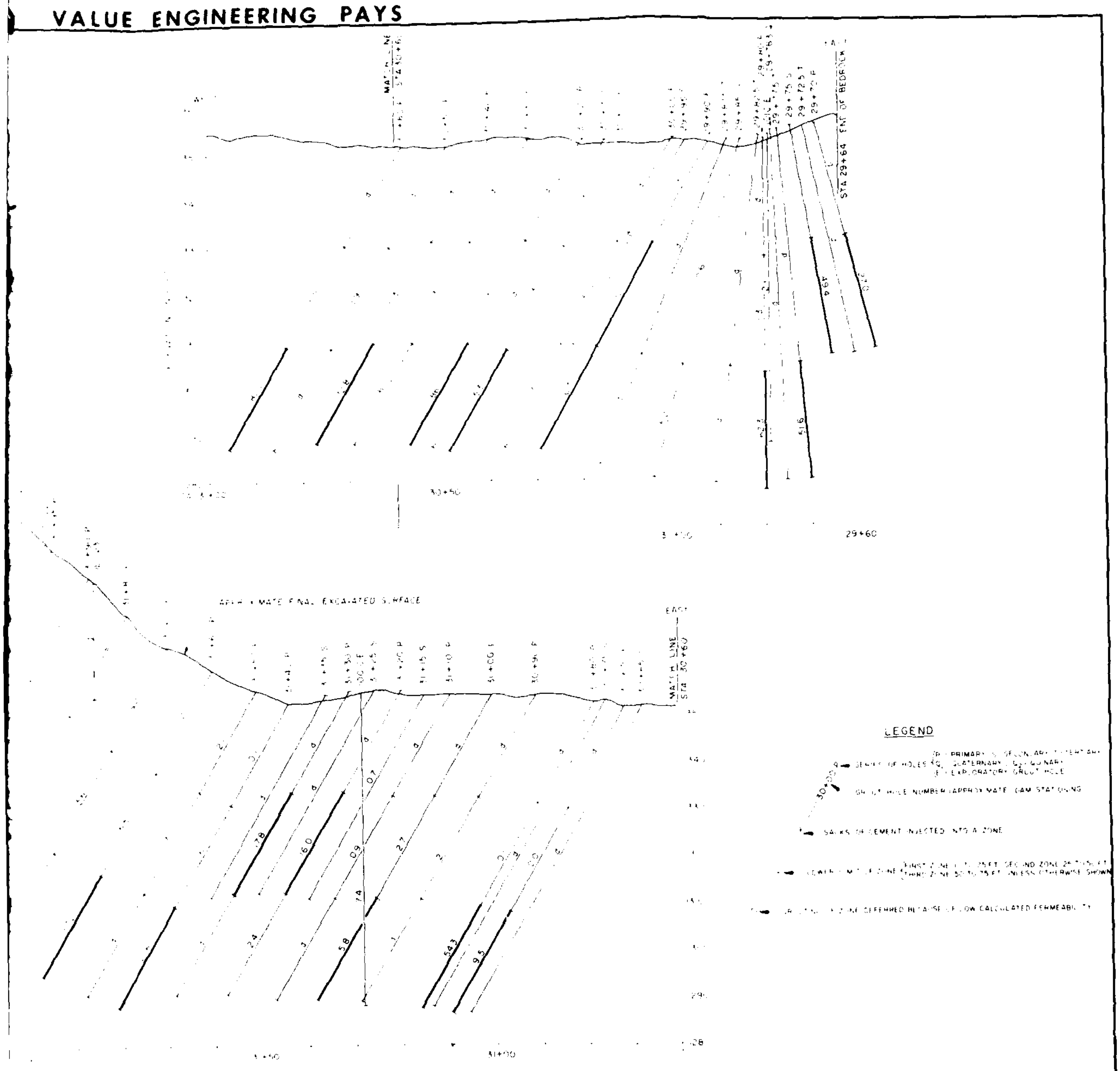
U.S. ARMY ENGINEER DISTRICT
LOS ANGELES CORPS OF ENGINEERS

VALUE ENGINEERING PAYS



SAFETY PAYS

VALUE ENGINEERING PAYS



[illegible] PROFILE (CORE ZONE)

NOTES

1. ALL GROUT HOLES WERE PERCUSSION DRILLED USING AN [illegible]
2. ALL EXPLORATORY GROUT HOLES WERE ROTARY DRILLED USING A [illegible] CORE BARREL.
3. CORE HOLES [illegible] 22 AND [illegible] WERE DRILLED DURING THE DESIGN PHASE PRIOR TO DAM CONSTRUCTION.
4. DETAILED PROFILES [illegible] AREAS OF SIGNIFICANT GROUT TAKE.
5. SEE TABLE [illegible] FOR COMPLETE GROUT HOLE INFORMATION, INCLUDING EACH HOLE LOCATION, WATER PRESSURE TEST DATA, [illegible] PRESSURE, AND MIX RATIO.
6. SEE PLATES 29 THRU 34 FOR GROUT HOLE PLAN.
7. [illegible] ZONE WHERE GROUT TAKES EXCEEDED [illegible] PER ZONE AND [illegible] SPACING REQUIRED.
8. STAGE GROUTING [illegible] HOLES WAS PERFORMED WHENEVER [illegible] WATER [illegible] OCCURRED. IT WAS ASSUMED THAT THE MAJORITY OF GROUT [illegible] STAGE OCCURRED AT OR NEAR THE [illegible] THEREFORE THE GROUT TAKE [illegible] LOCALIZED INTERVAL.
9. FOR PERCUSSION DRILLED GROUT HOLES, GROUT TAKES WERE ASSUMED TO BE EVENLY DISTRIBUTED WITHIN EACH ZONE.
10. SEE FIGURES [illegible] THRU 7 FOR GEOLOGIC LOGS OF EXPLORATORY GROUT CORE HOLES.

GILA RIVER BASIN
PHOENIX ARIZONA AND VICINITY (INCLUDING NEW RIVER)

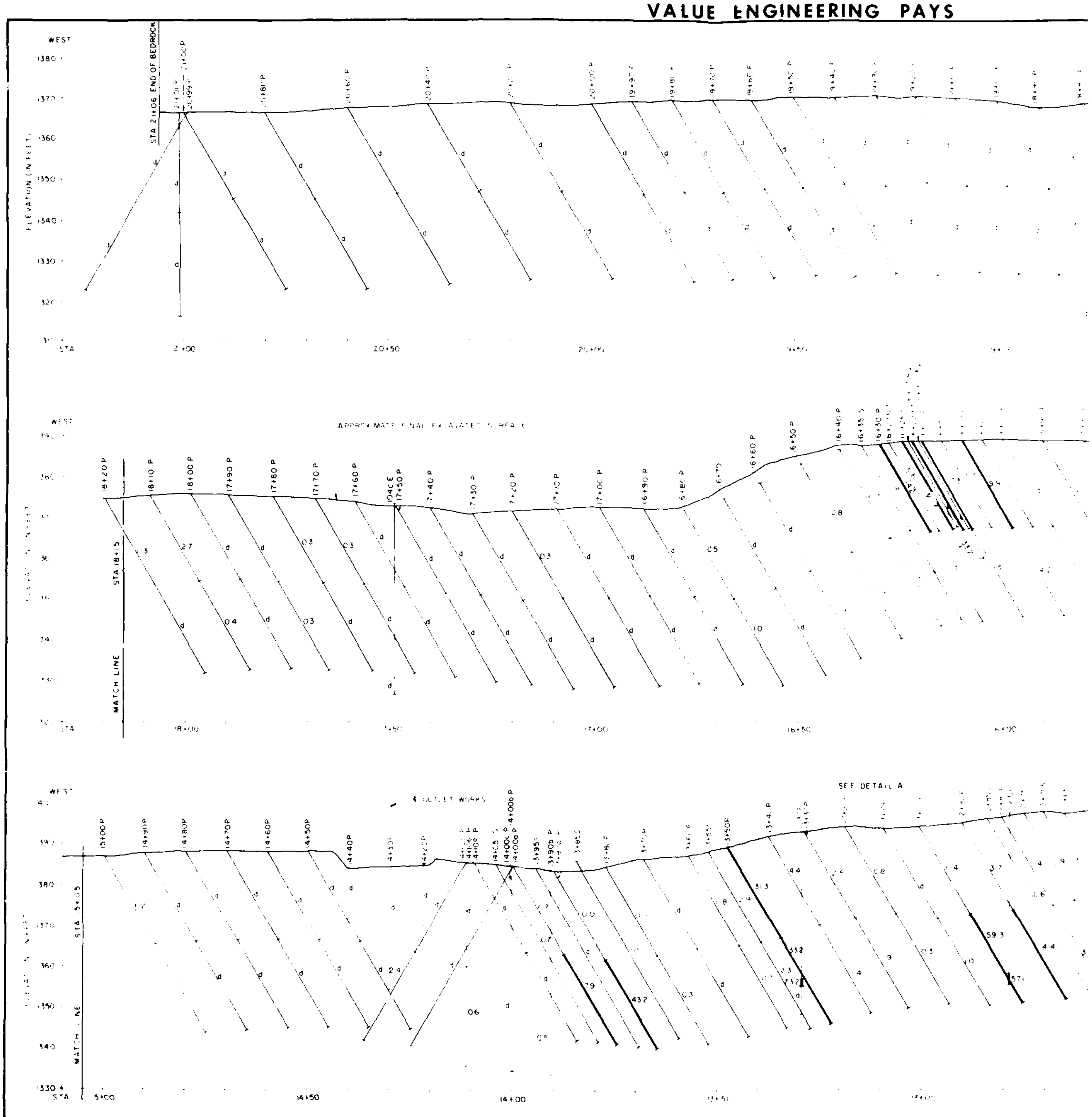
NEW RIVER DAM

FOUNDATION GROUTING PROFILE
STA 33+27 TO STA 29+64

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

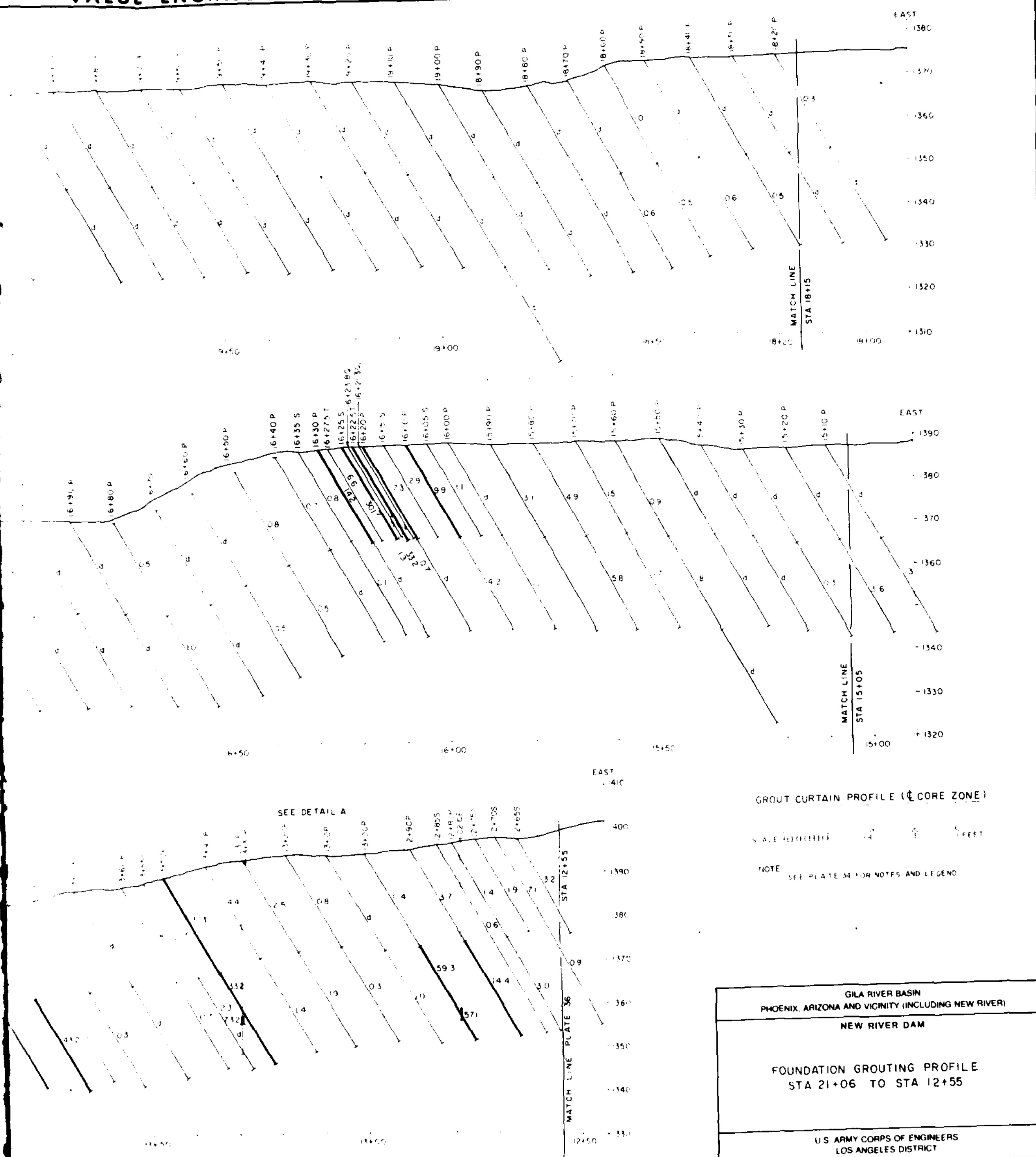
SAFETY PAYS

VALUE ENGINEERING PAYS



SAFETY PAYS

VALUE ENGINEERING PAYS



GILA RIVER BASIN
PHOENIX, ARIZONA AND VICINITY (INCLUDING NEW RIVER)
NEW RIVER DAM

FOUNDATION GROUTING PROFILE
STA 21+06 TO STA 12+55

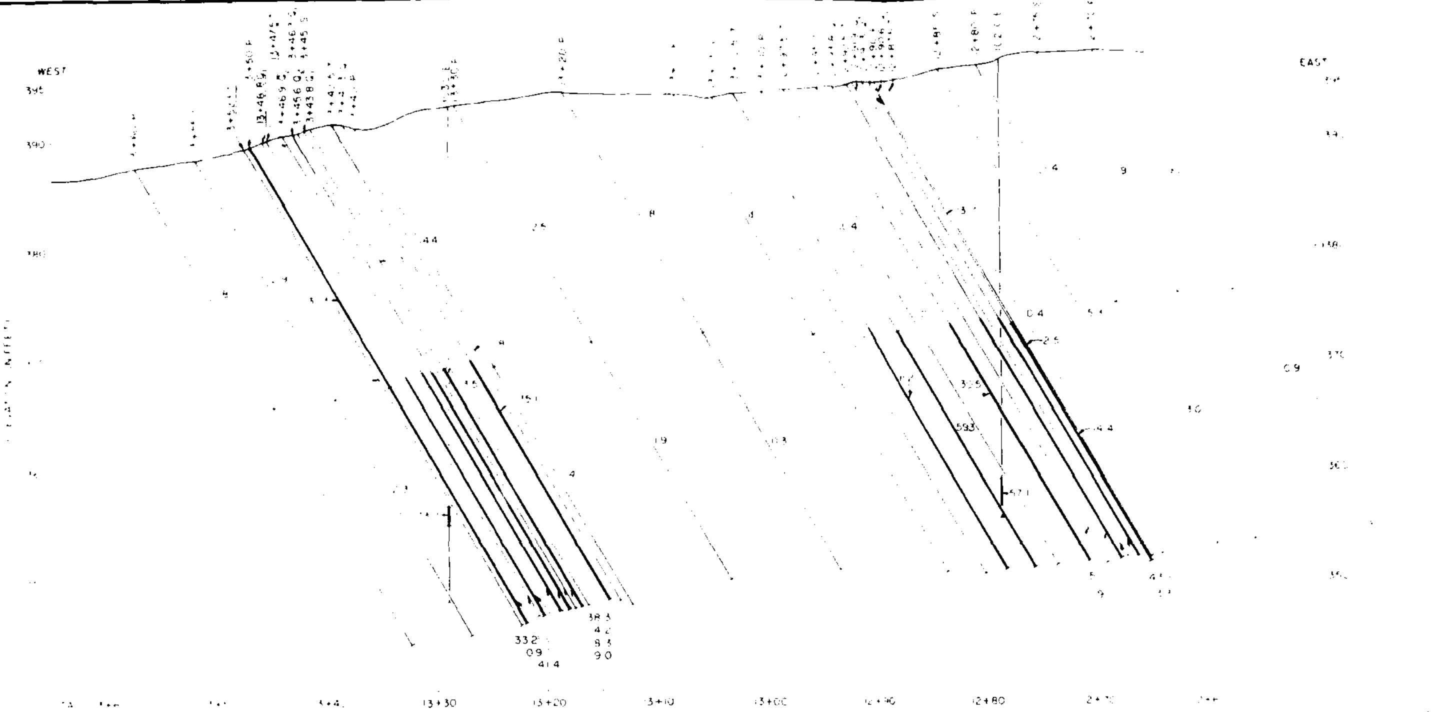
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

FOUNDATION REPORT

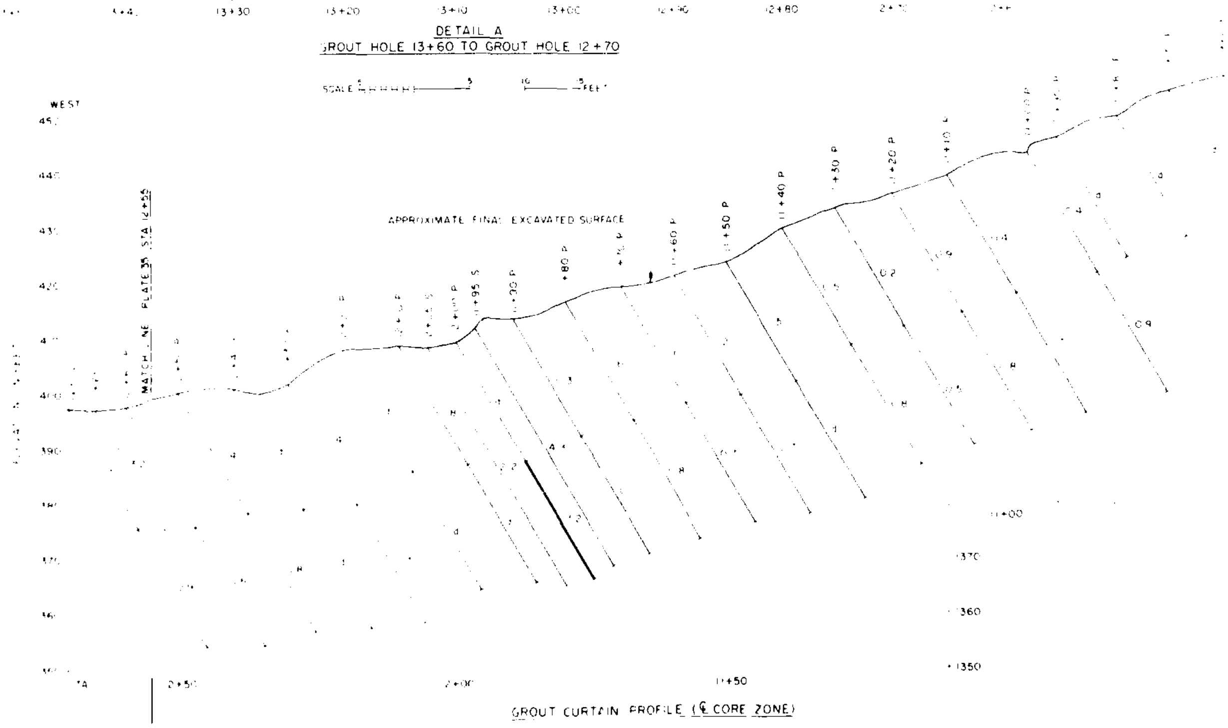
PLATE 35

SAFETY PAYS

VALUE ENGINEERING PAYS



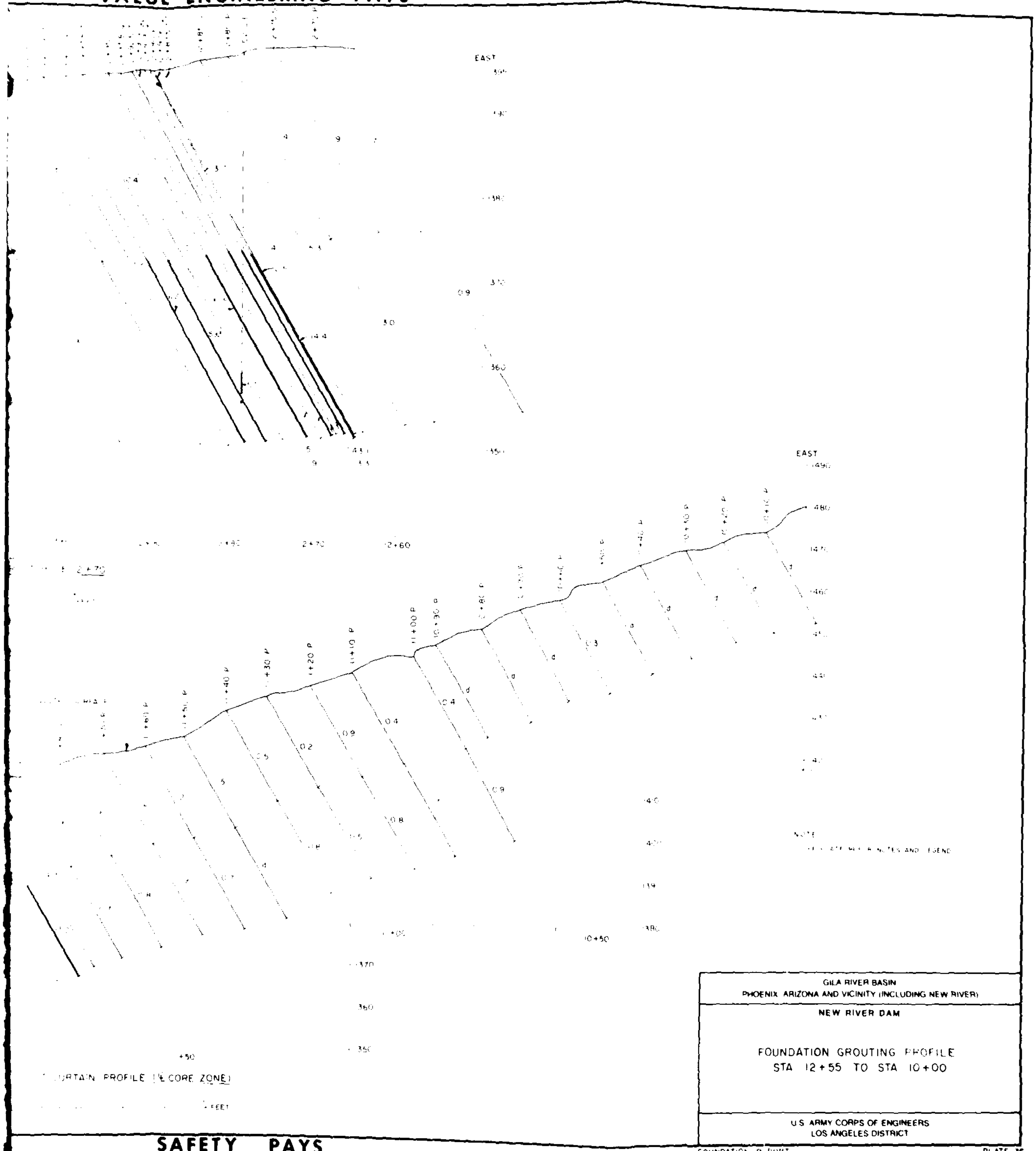
DETAIL A
GROUT HOLE 13+60 TO GROUT HOLE 12+70



GROUT CURTAIN PROFILE (℄ CORE ZONE)

SAFETY PAYS

VALUE ENGINEERING PAYS

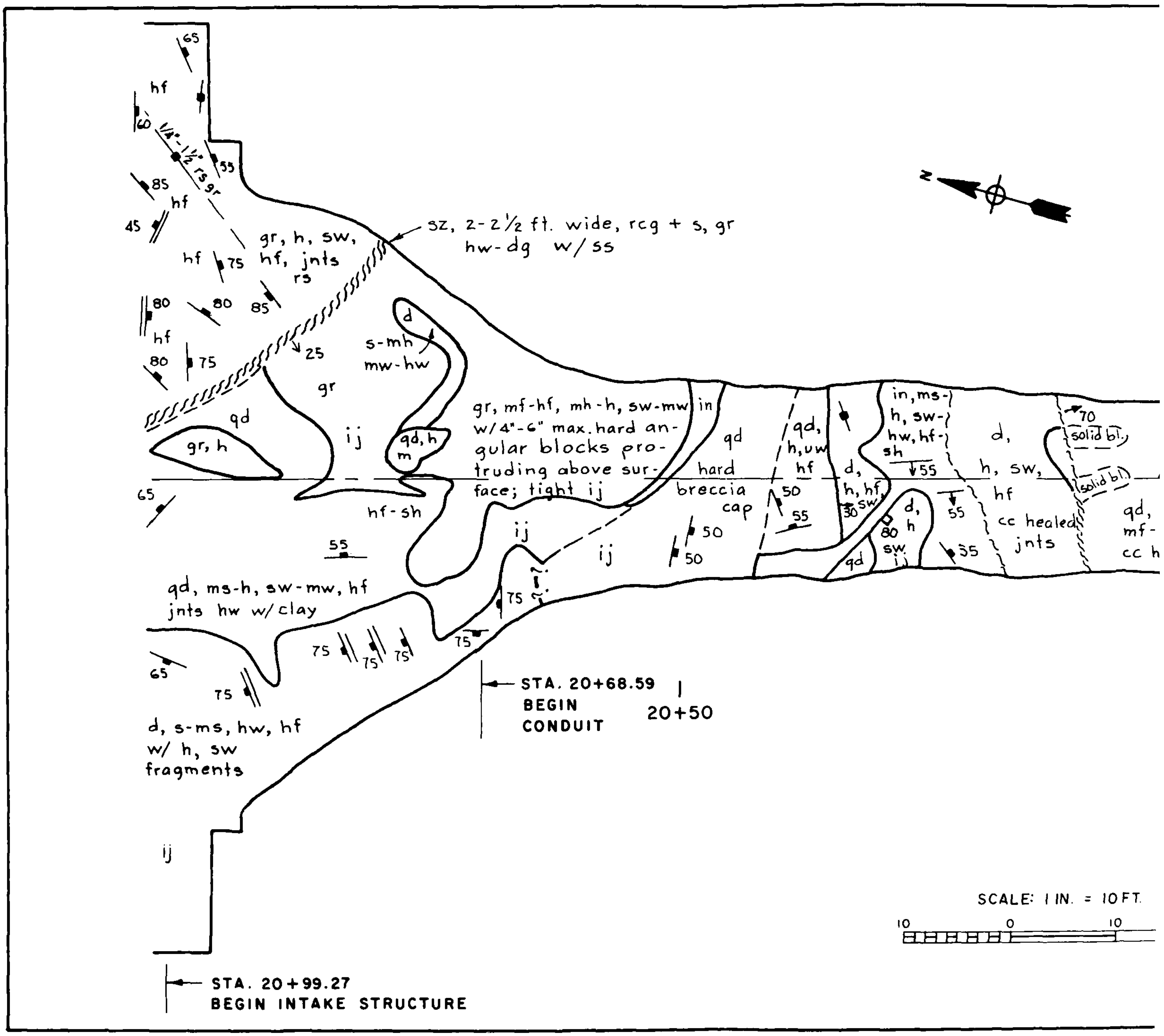


GILA RIVER BASIN
PHOENIX ARIZONA AND VICINITY (INCLUDING NEW RIVER)
NEW RIVER DAM

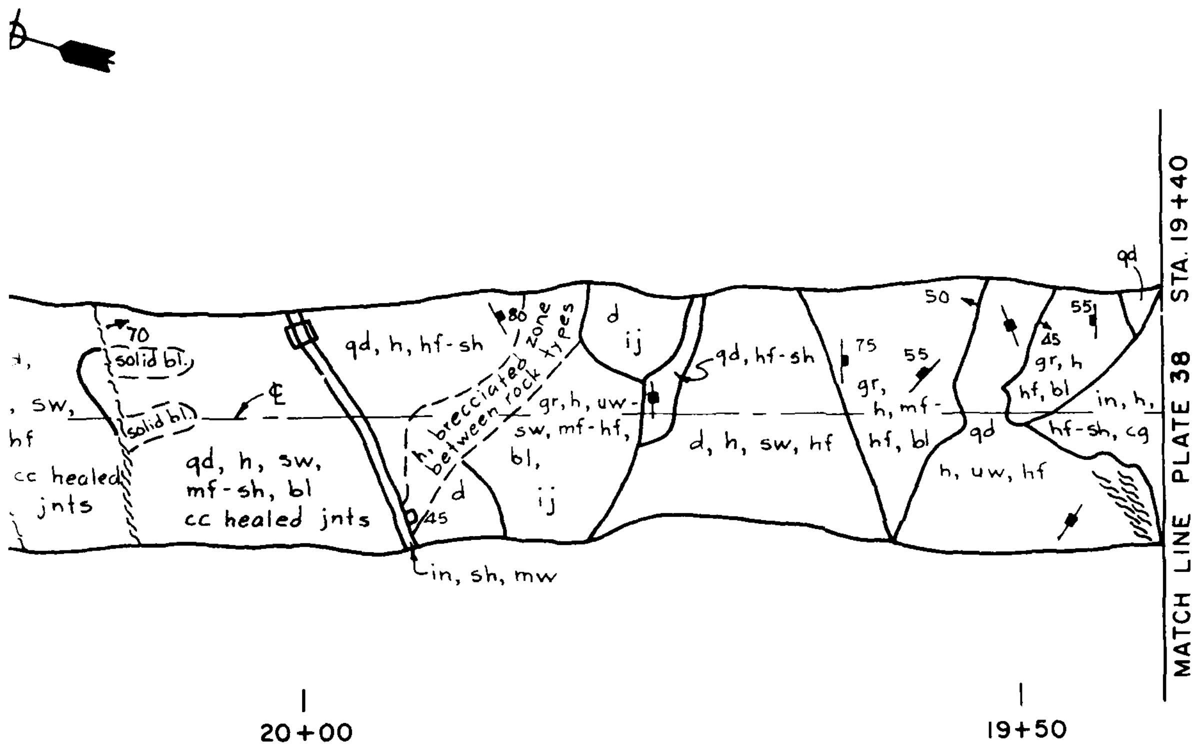
FOUNDATION GROUTING PROFILE
STA 12+55 TO STA 10+00

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

SAFETY PAYS



N
hf
1/4"-1 1/2" rs gr
gr, h, sw, hf, jnts rs
sz, 2-2 1/2 ft. wide, rcg + s, gr hw-dg w/ ss
d
s-mh mw-hw
gr
qd
gr, h
ij
qd, h m
gr, mf-hf, mh-h, sw-mw w/4"-6" max. hard angular blocks protruding above surface; tight ij
in
qd hard breccia cap
qd, h, uw hf
d, h, hf, sw
in, ms-h, sw-hw, hf-sh
d, h, sw, hf
cc healed jnts
solid bl.
qd, mf-cc h
d, h sw ij
qd
hf-sh
qd, ms-h, sw-mw, hf jnts hw w/clay
d, s-ms, hw, hf w/ h, sw fragments
ij
STA. 20+68.59 BEGIN CONDUIT
20+50
STA. 20+99.27 BEGIN INTAKE STRUCTURE
SCALE: 1 IN. = 10 FT.
10 0 10



ALE: 1 IN. = 10 FT.

0 10 20

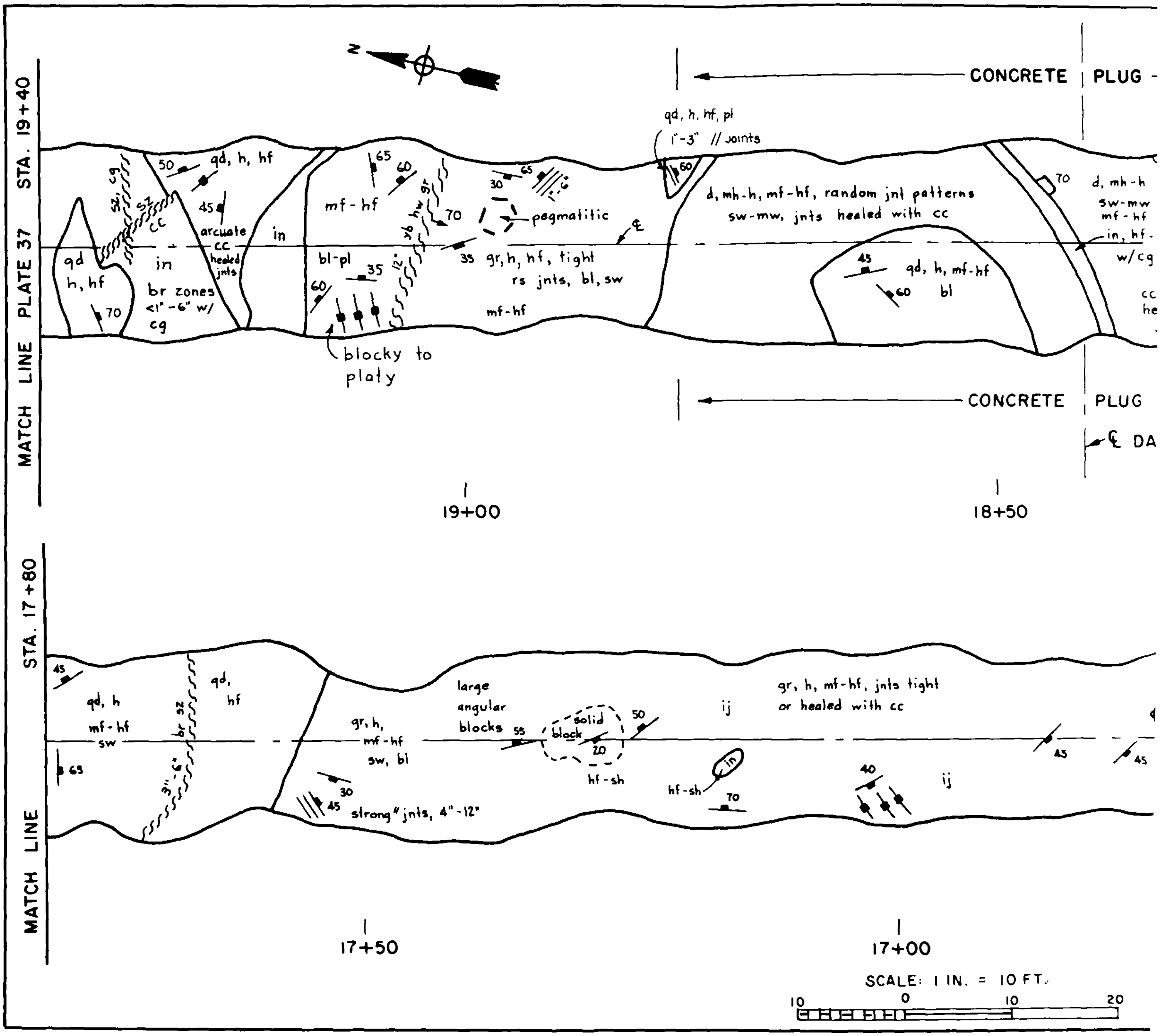
NOTE:

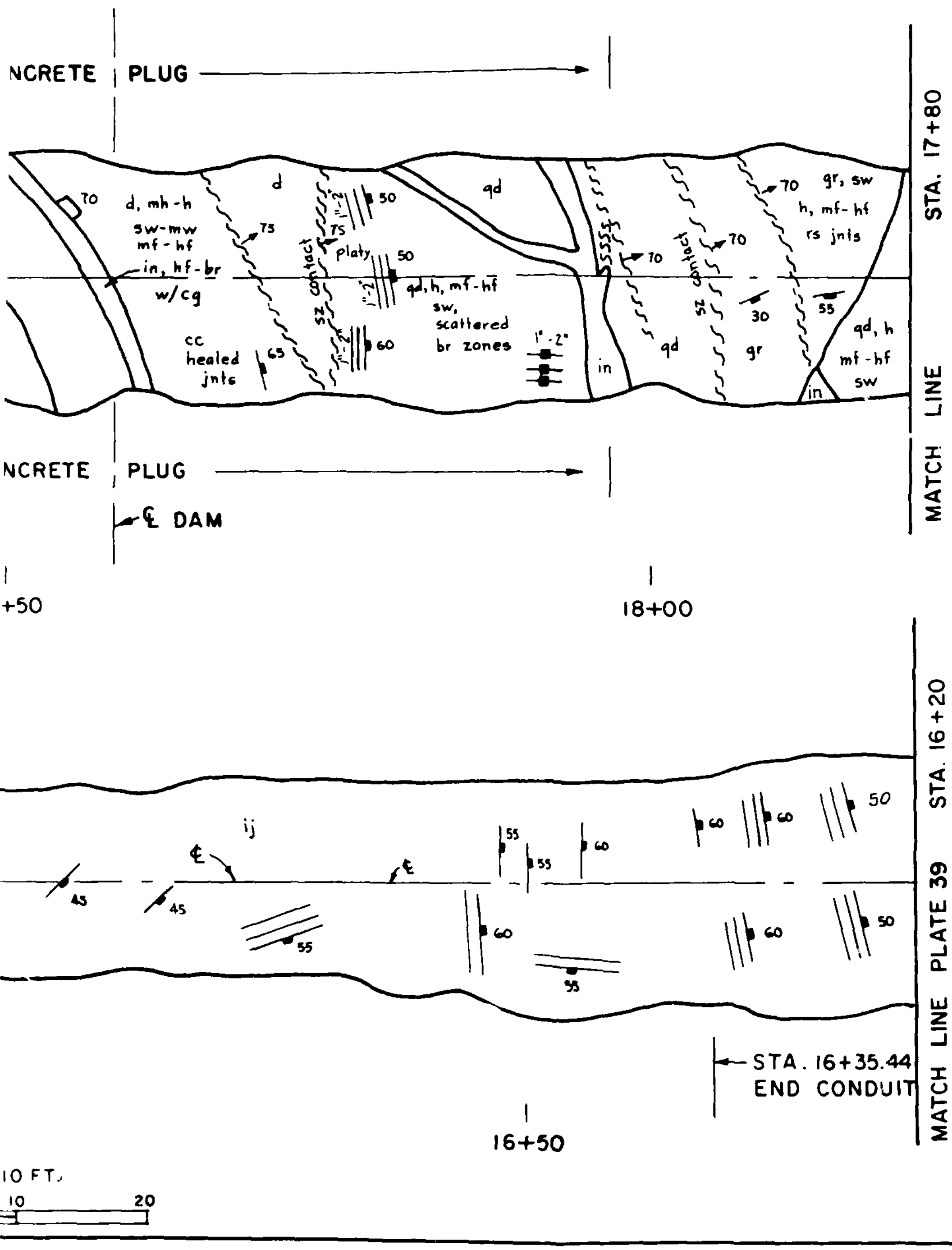
SEE PLATE 28 FOR LEGEND AND GENERAL NOTES.

GILA RIVER BASIN
PHOENIX AZ. AND VICINITY (INCL. NEW RIVER)

NEW RIVER DAM
OUTLET WORKS
FOUNDATION GEOLOGY
STA. 20+99 TO STA. 19+40

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES CORPS OF ENGINEERS


MATCH LINE PLATE 37 STA. 19+40
CONCRETE PLUG
qd, h, hf, pl
1"-3" // joints
qd, h, hf
qd
h, hf
in
br zones
<1"-6" w/
cg
arcuate
cc
healed
jnts
mf-hf
bl-pl
pegmatitic
gr, h, hf, tight
rs jnts, bl, sw
mf-hf
blocky to
platy
d, mh-h, mf-hf, random jnt patterns
sw-mw, jnts healed with cc
qd, h, mf-hf
bl
d, mh-h
sw-mw
mf-hf
in, hf-
w/cg
CONCRETE PLUG
℄ DA
19+00
18+50
MATCH LINE STA. 17+80
qd, h
mf-hf
sw
qd,
hf
br sz
3"-6"
gr, h,
mf-hf
sw, bl
large
angular
blocks
solid
block
hf-sh
strong" jnts, 4"-12"
ij
gr, h, mf-hf, jnts tight
or healed with cc
hf-sh
in
17+50
17+00
SCALE: 1 IN. = 10 FT.
10
0
10
20



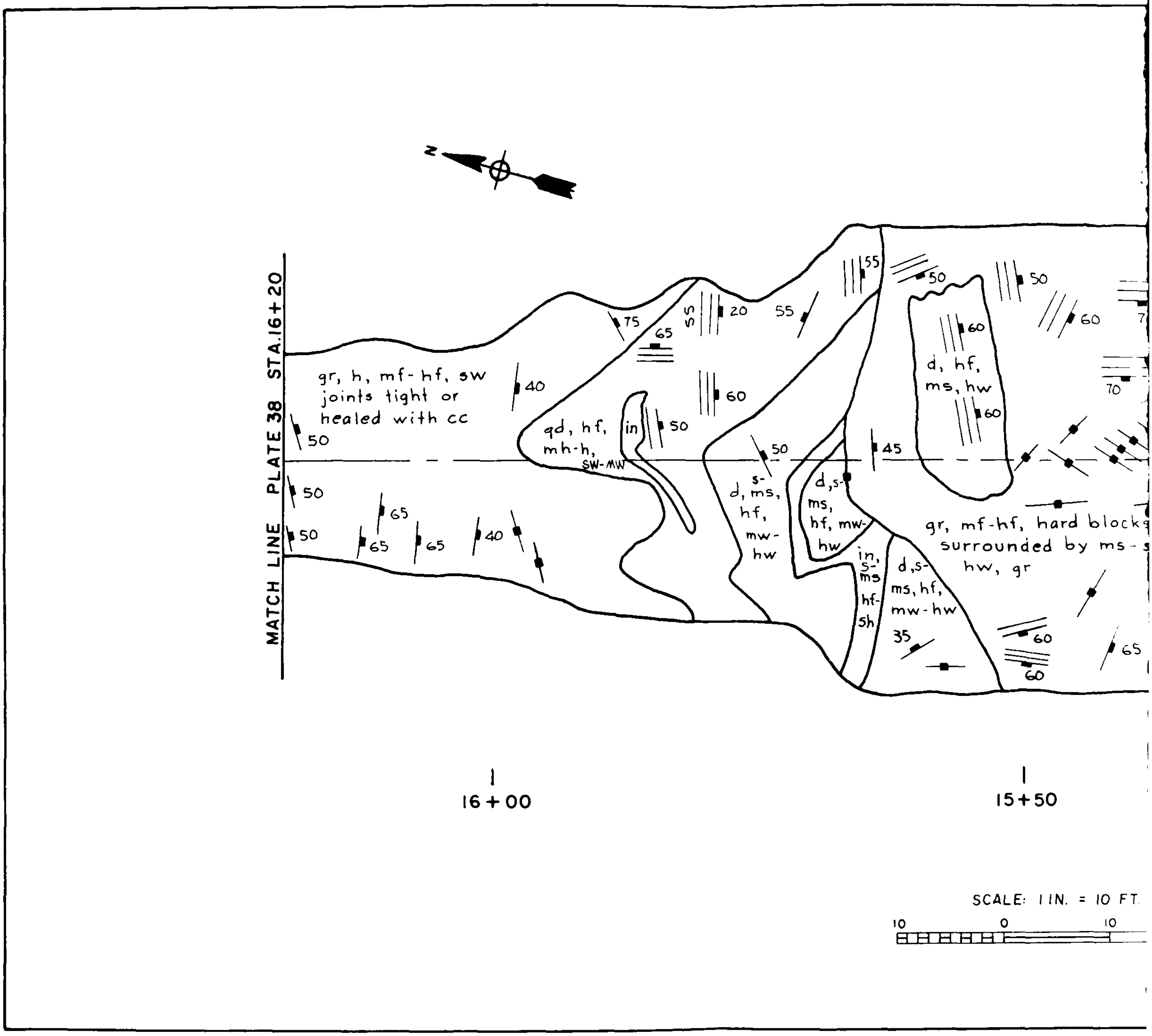
NOTE:

SEE PLATE 28 FOR LEGEND AND GENERAL NOTES.

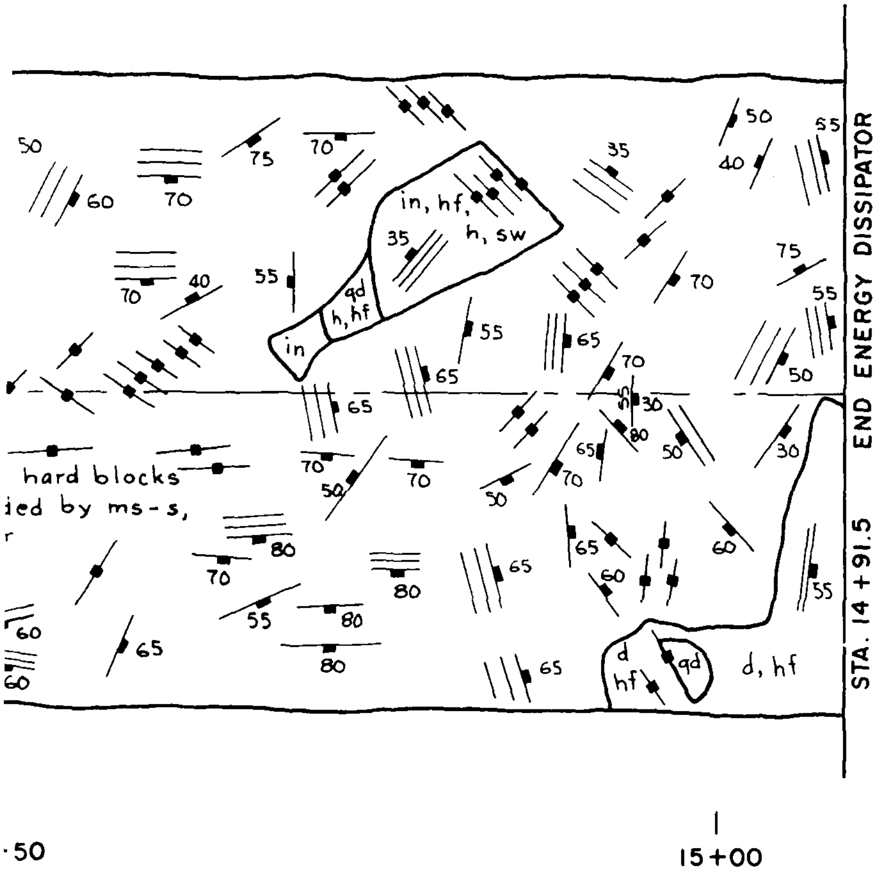
GILA RIVER BASIN
PHOENIX, AZ. AND VICINITY (INCL. NEW RIVER)

NEW RIVER DAM
OUTLET WORKS
FOUNDATION GEOLOGY
STA. 19+40 TO STA. 16+20

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES CORPS OF ENGINEERS



MATCH LINE PLATE 38 STA.16+20
gr, h, mf-hf, sw joints tight or healed with cc
qd, hf, mh-h, sw-mw
in
d, s-ms, hf, mw-hw
d, s-ms, hf, mw-hw
in, s-ms hf-sh
d, s-ms, hf, mw-hw
d, hf, ms, hw
gr, mf-hf, hard blocks surrounded by ms-s hw, gr
16+00
15+50
SCALE: 1 IN. = 10 FT.
10
0
10



1 IN. = 10 FT.

10 20

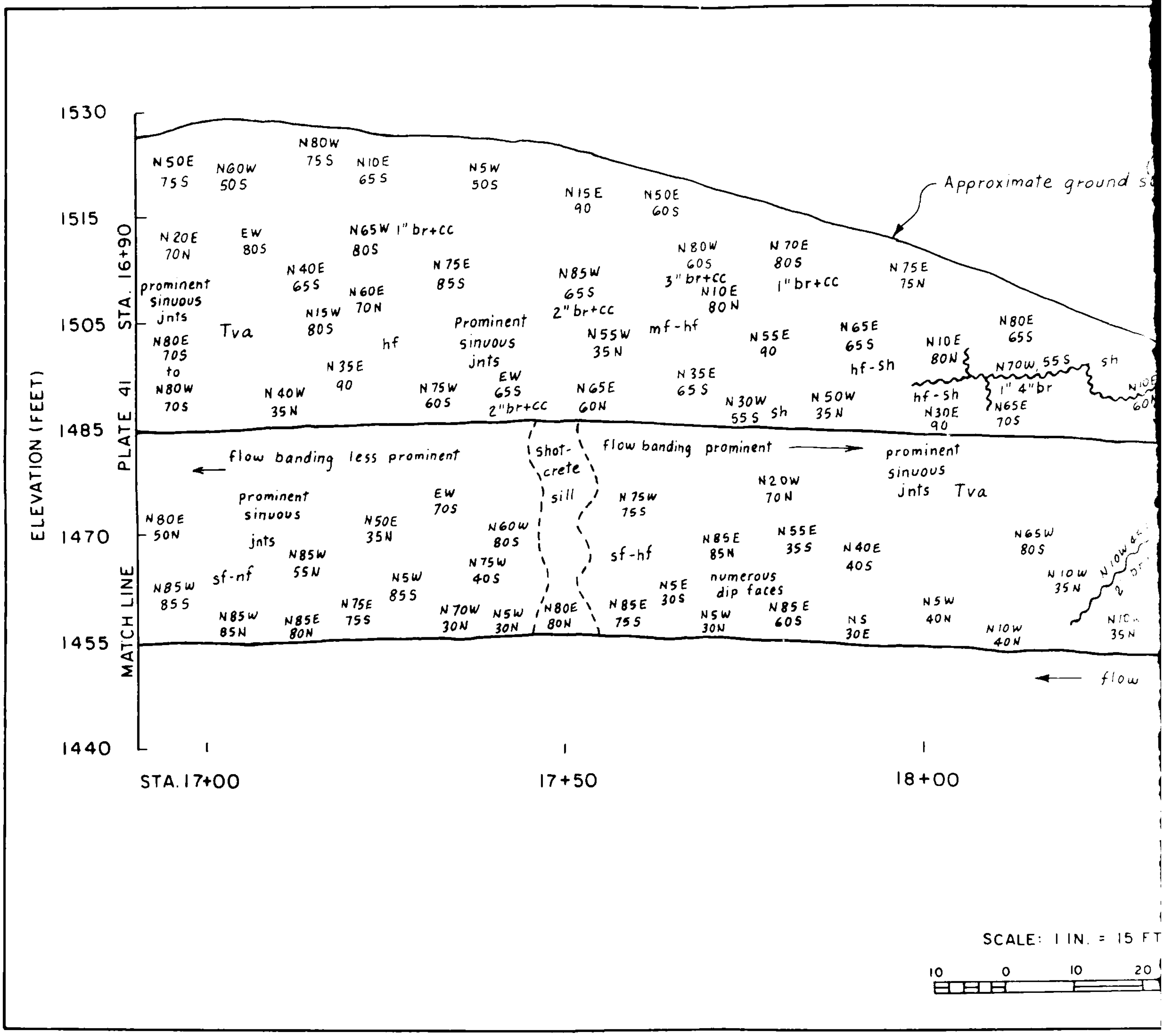
NOTE:

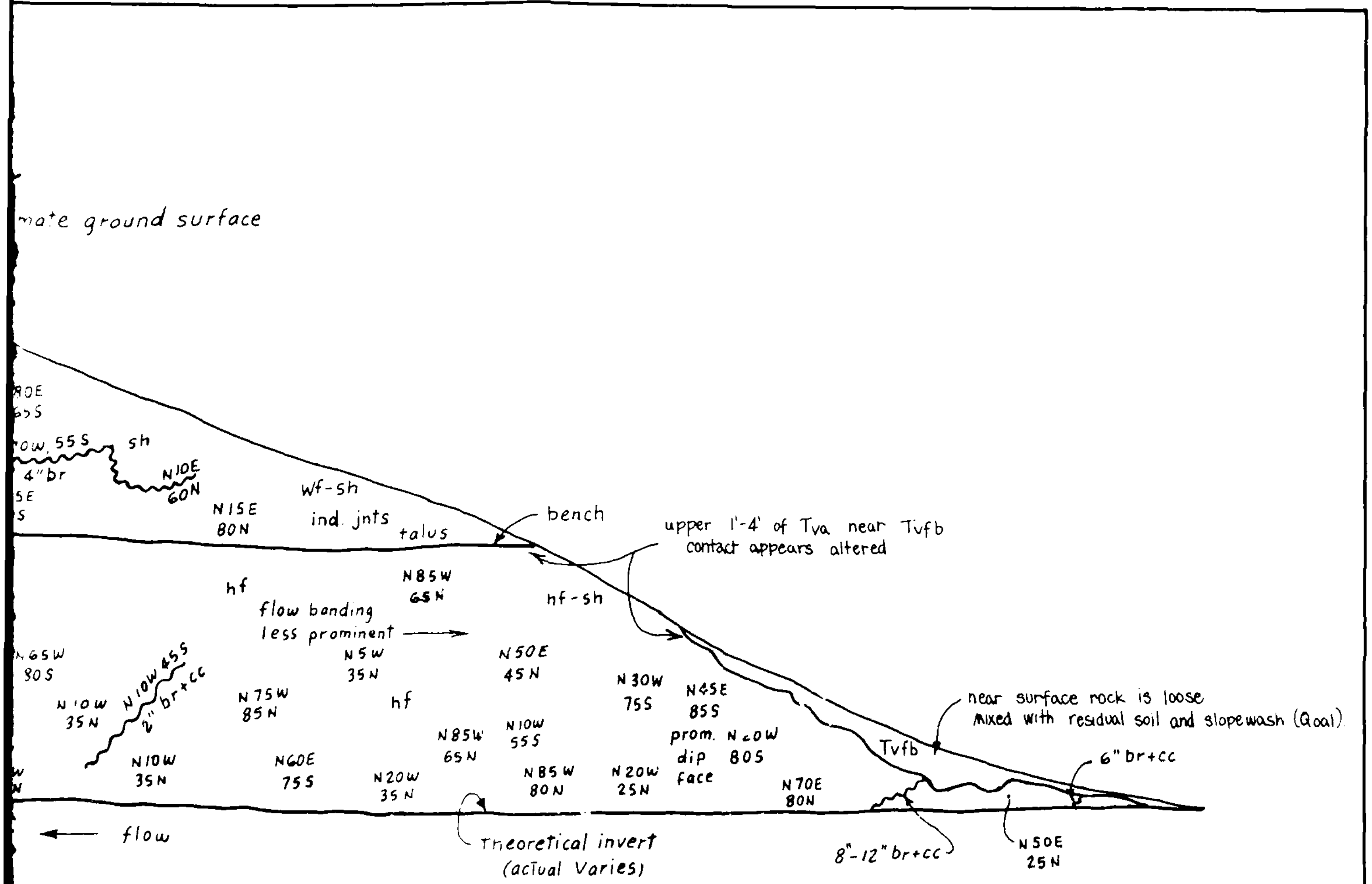
SEE PLATE 28 FOR LEGEND AND GENERAL NOTES.

GILA RIVER BASIN
PHOENIX, AZ AND VICINITY (INCL. NEW RIVER)

NEW RIVER DAM
OUTLET WORKS
FOUNDATION GEOLOGY
STA. 16+20 TO STA. 14+91

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES CORPS OF ENGINEERS


ELEVATION (FEET)
1530
1515
1505
1485
1470
1455
1440
STA. 16+90
PLATE 41
MATCH LINE
STA. 17+00
17+50
18+00
Approximate ground s
N 50E 75 S
N60W 50 S
N80W 75 S
N10E 65 S
N5W 50S
N15E 90
N50E 60S
N 20E 70N
EW 80S
N65W 1" br+cc 80S
N40E 65S
N60E 70N
N75E 85S
N85W 65S 2" br+cc
N80W 60S 3" br+cc
N10E 80N
N 70E 80S 1" br+cc
N 75E 75N
prominent sinuous jnts
N15W 80S
Tva
hf
Prominent sinuous jnts
N55W 35N
mf-hf
N55E 90
N65E 65S
hf-sh
N10E 80N
N80E 65S
N80E 70S to N80W 70S
N35E 90
N40W 35N
N75W 60S
EW 65S 2" br+cc
N65E 60N
N35E 65S
N30W 55S
sh
N50W 35N
hf-sh
N30E 90
N70W, 55S
1" 4" br
N65E 70S
sh
flow banding less prominent
Shot-crete sill
flow banding prominent
prominent sinuous jnts
Tva
N20W 70N
N80E 50N
prominent sinuous jnts
N50E 35N
EW 70S
N60W 80S
N75W 75S
N85E 85N
N55E 35S
N40E 40S
N65W 80S
N85W 55N
N75W 40S
sf-hf
sf-nf
N85W 85S
N5W 85S
N5E 30S
numerous dip faces
N10W 35N
N85W 85N
N85E 80N
N75E 75S
N70W 30N
N5W 30N
N80E 80N
N85E 75S
N5W 30N
N85E 60S
NS 30E
N5W 40N
N10W 40N
flow
SCALE: 1 IN. = 15 FT
10
0
10
20



18+50 19+00 19+50

NOTES:

1. SEE PLATE 28 FOR LEGEND AND GENERAL NOTES.
2. THE GEOLOGY EXPOSED ON THE SOUTH (LEFT) SIDE OF THE SPILLWAY EXCAVATION IS ESSENTIALLY THE SAME AS THAT ON THE NORTH SIDE.
3. ALLUVIUM (QOAL) IS GENERALLY NOT DESIGNATED WHERE THE THICKNESS IS LESS THAN ONE FOOT.

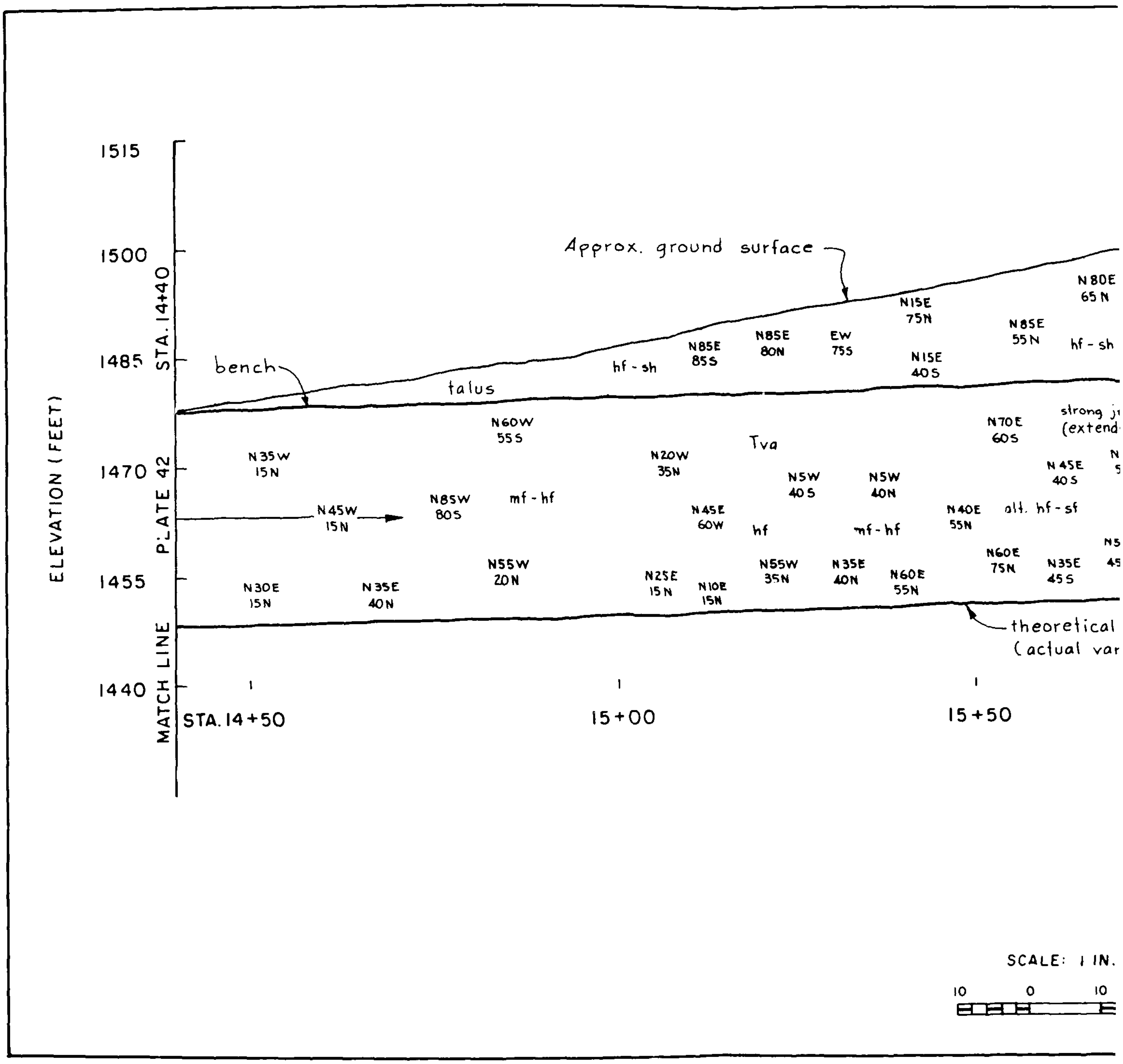
LE: 1 IN. = 15 FT.

10 20 30

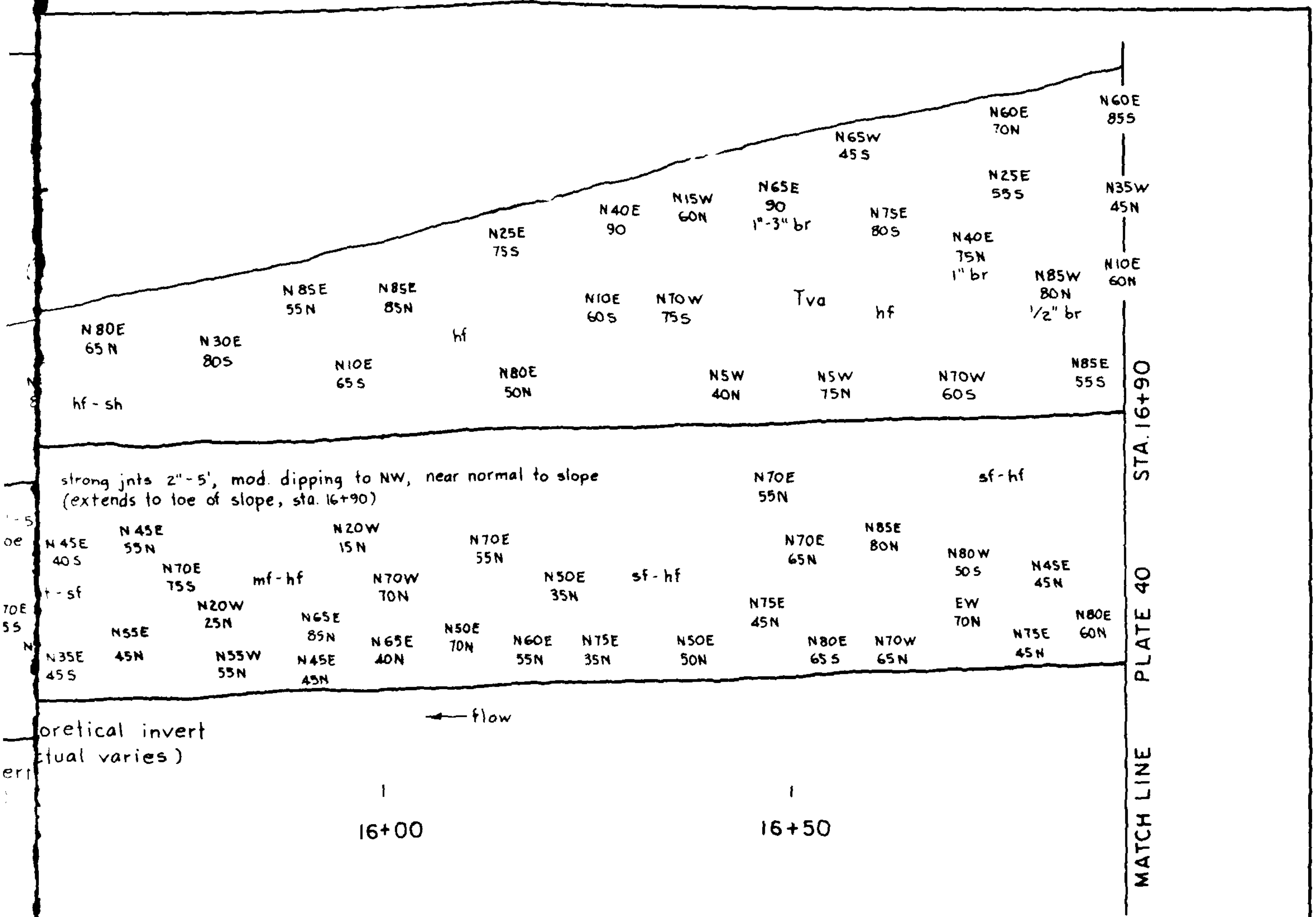
GILA RIVER BASIN
PHOENIX AZ. AND VICINITY (INCL. NEW RIVER)

NEW RIVER DAM
SPILLWAY
GEOLOGY OF NORTH (RIGHT) WALL
STA. 19+50 TO STA. 15+90

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES CORPS OF ENGINEERS



ELEVATION (FEET)
1515
1500
1485
1470
1455
1440
STA. 14+40
PLATE 42
MATCH LINE
Approx. ground surface
bench
talus
hf - sh
N85E
85S
N85E
80N
EW
75S
N15E
75N
N15E
40S
N85E
55N
N80E
65N
hf - sh
N60W
55S
Tva
N70E
60S
N35W
15N
N20W
35N
N5W
40S
N5W
40N
N45E
40S
N45W
15N
N85W
80S
mf - hf
N45E
60W
hf
mf - hf
N40E
55N
alt. hf - sf
N55W
20N
N55W
35N
N35E
40N
N60E
55N
N60E
75N
N35E
45S
N30E
15N
N35E
40N
N25E
15N
N10E
15N
theoretical
(actual var
STA. 14+50
15+00
15+50
SCALE: 1 IN.
10
0
10



NOTES:

1. SEE PLATE 28 FOR LEGEND AND GENERAL NOTES.
2. SEE PLATE 40 FOR NOTES.

LE: 1 IN. = 15 FT.

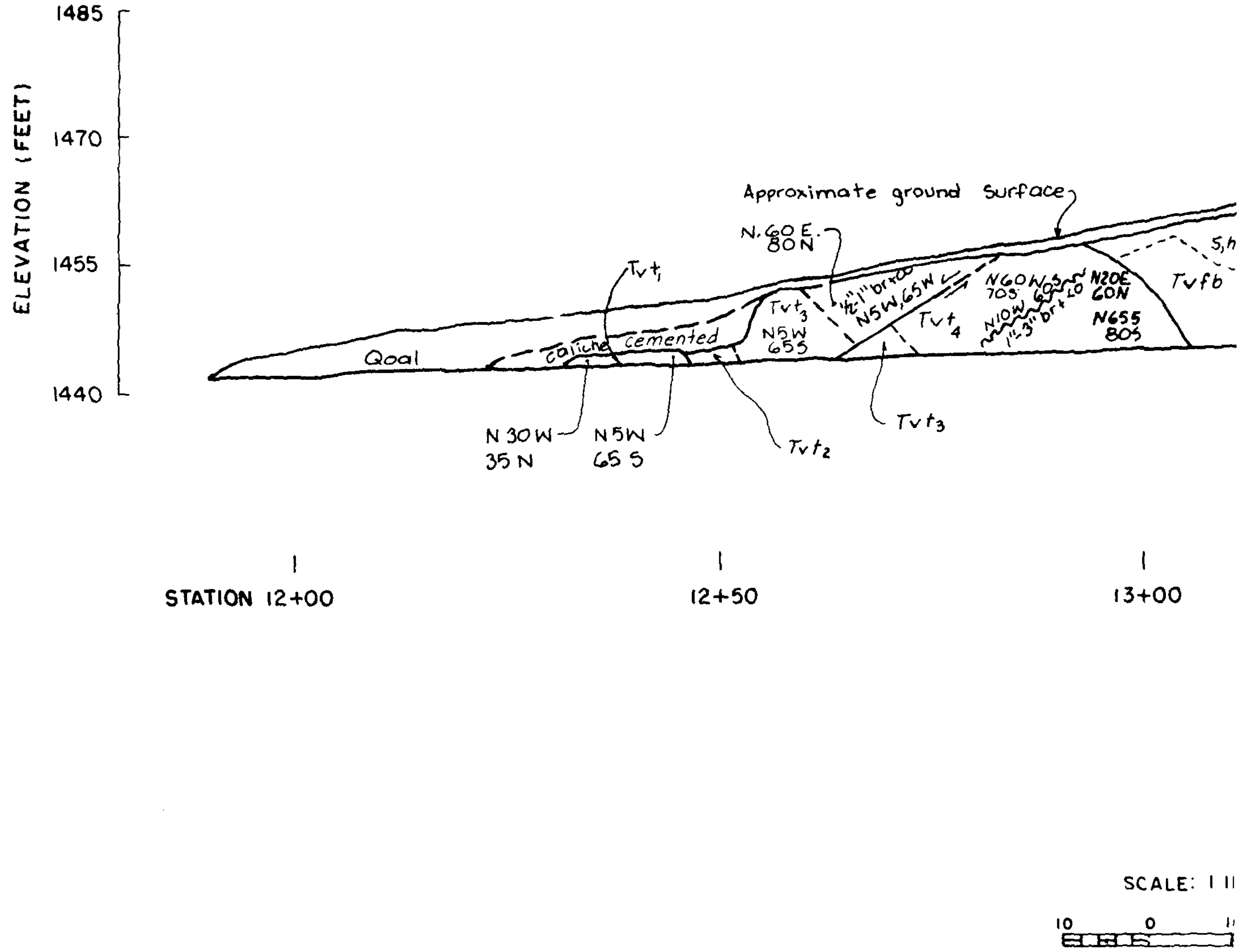
10 20 30

GILA RIVER BASIN
PHOENIX AZ. AND VICINITY (INCL. NEW RIVER)

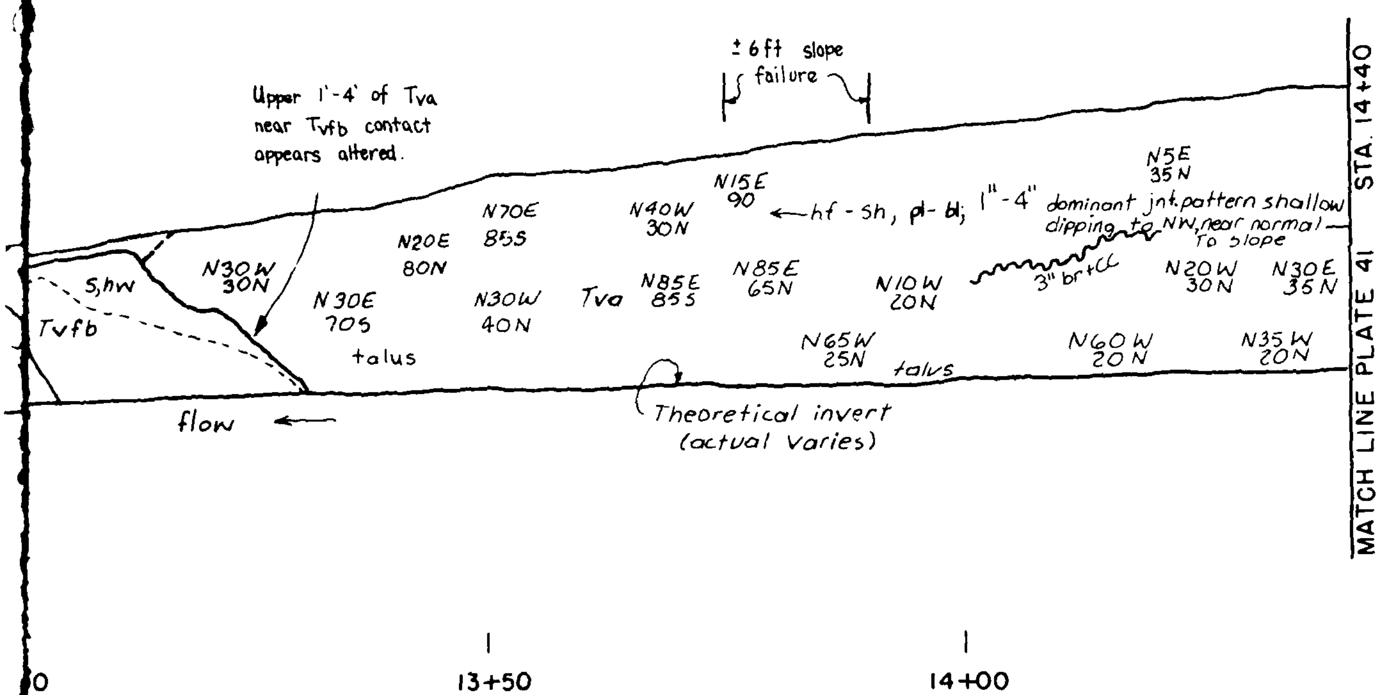
NEW RIVER DAM
SPILLWAY

GEOLOGY OF NORTH (RIGHT) WALL
STA. 16+90 TO STA. 14+40

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES CORPS OF ENGINEERS



ELEVATION (FEET)
1485
1470
1455
1440
Approximate ground surface
N. 60 E. 80 N
Tvt1
Tvt3
N5W 65S
N5W, 65W
Tvt4
N60W 70S
N10W
1"-3" br+co
N20E 60N
N65S 80S
Tvfb
Qoal
caliche cemented
N 30 W 35 N
N5W 65 S
Tvt2
Tvt3
STATION 12+00
12+50
13+00
SCALE: 1 I
10
0



NOTES:

1. SEE PLATE 28 FOR LEGEND AND GENERAL NOTES.
2. SEE PLATE 40 FOR NOTES.

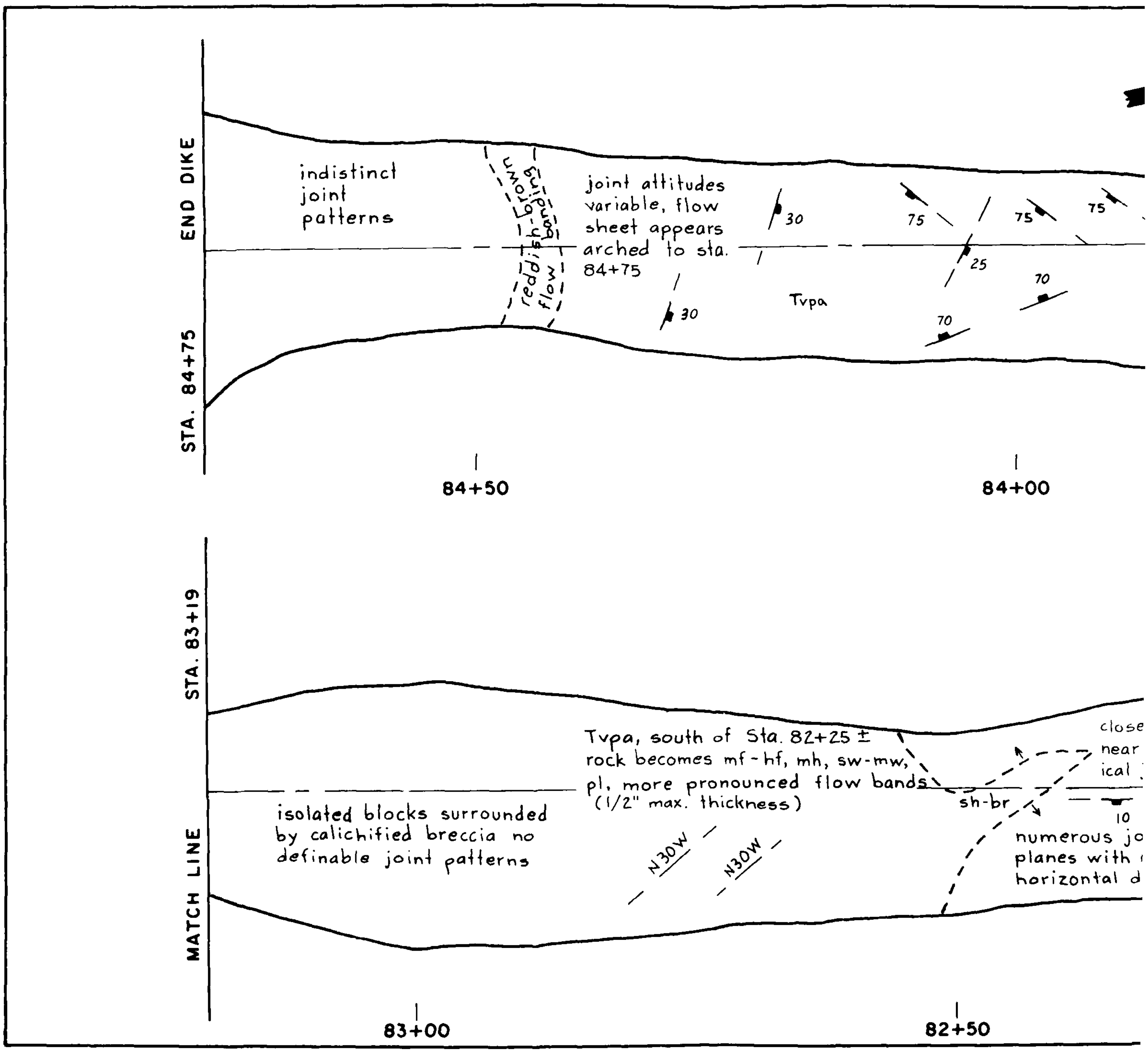
LE: 1 IN. = 15 FT.

10 20 30

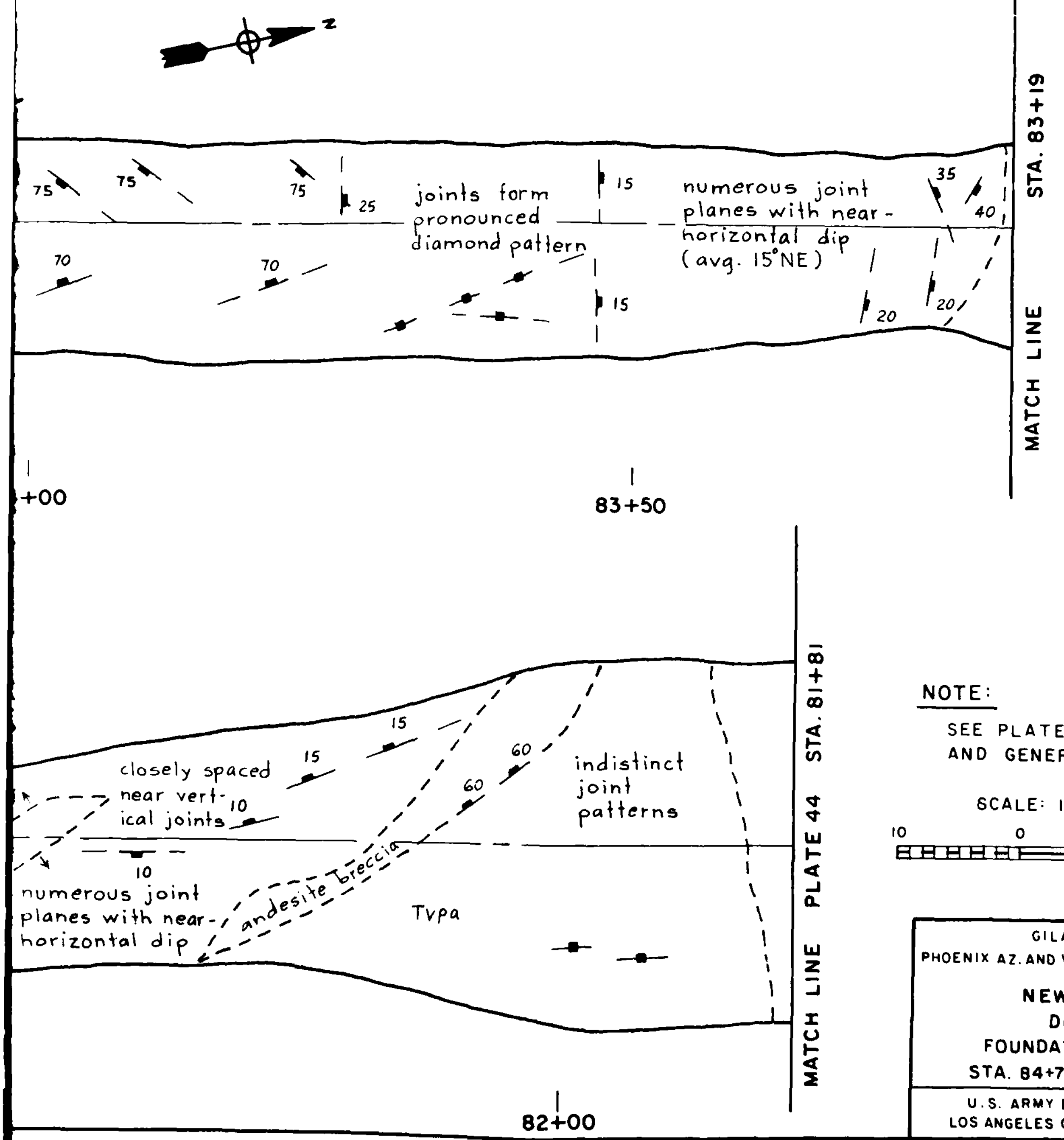
GILA RIVER BASIN
PHOENIX AZ. AND VICINITY (INCL. NEW RIVER)

NEW RIVER DAM
SPILLWAY
GEOLOGY OF NORTH (RIGHT) WALL
STA. 14+40 TO STA. 11+90

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES CORPS OF ENGINEERS



END DIKE
STA. 84+75
indistinct joint patterns
reddish-brown flow banding
joint attitudes variable, flow sheet appears arched to sta. 84+75
30
30
Tvpa
75
75
75
25
70
70
84+50
84+00
STA. 83+19
MATCH LINE
Tvpa, south of Sta. 82+25 ± rock becomes mf-hf, mh, sw-mw, pl, more pronounced flow bands (1/2" max. thickness)
isolated blocks surrounded by calichified breccia no definable joint patterns
N30W
N30W
sh-br
10
close
near
ical
numerous jo
planes with
horizontal d
83+00
82+50



NOTE:

SEE PLATE 28 FOR LEGEND AND GENERAL NOTES.

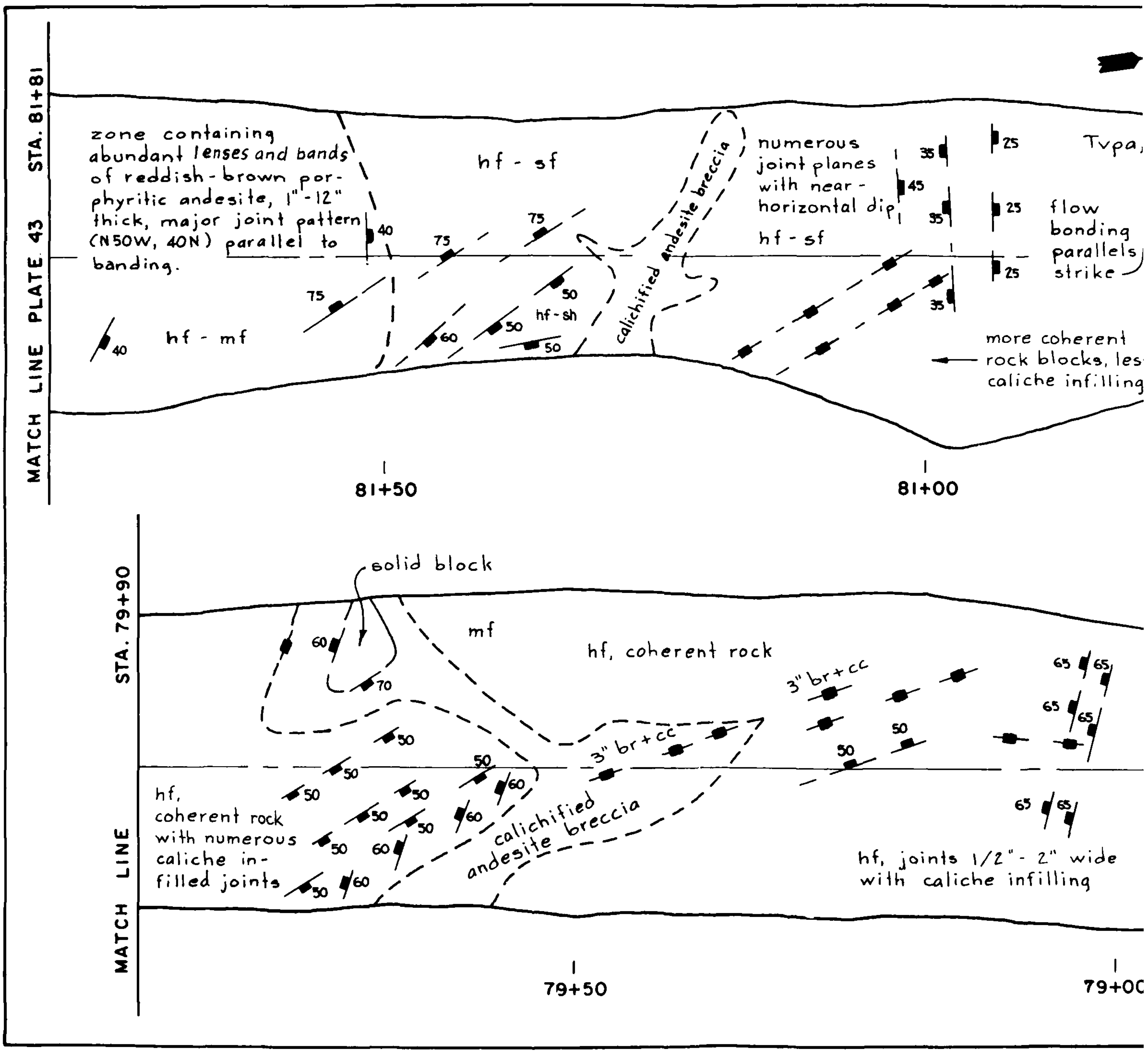
SCALE: 1 IN. = 10 FT.

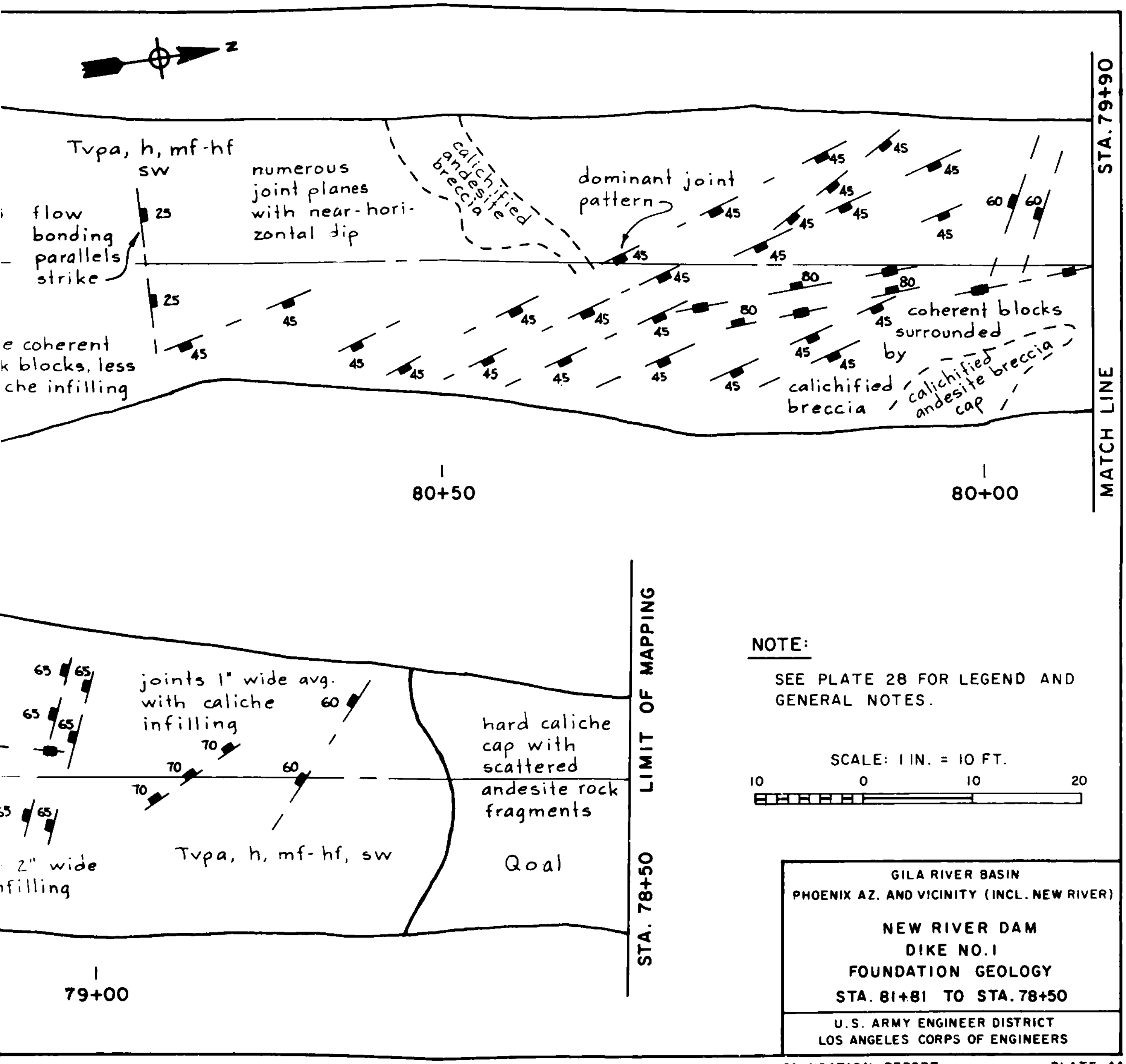
10 0 10 20

GILA RIVER BASIN
PHOENIX AZ. AND VICINITY (INCL. NEW RIVER)

NEW RIVER DAM
DIKE NO. 1
FOUNDATION GEOLOGY
STA. 84+75 TO STA. 81+81

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES CORPS OF ENGINEERS


MATCH LINE PLATE 43 STA. 81+81
zone containing abundant lenses and bands of reddish-brown porphyritic andesite, 1"-12" thick, major joint pattern (N50W, 40N) parallel to banding.
hf - sf
hf - mf
hf - sh
calichified andesite breccia
numerous joint planes with near-horizontal dip
hf - sf
Tvpa,
flow bonding parallels strike
more coherent rock blocks, les caliche infilling
81+50
81+00
MATCH LINE STA. 79+90
solid block
mf
hf, coherent rock
3" br + cc
3" br + cc
hf, coherent rock with numerous caliche in-filled joints
calichified andesite breccia
hf, joints 1/2" - 2" wide with caliche infilling
79+50
79+00



NOTE:

SEE PLATE 28 FOR LEGEND AND GENERAL NOTES.

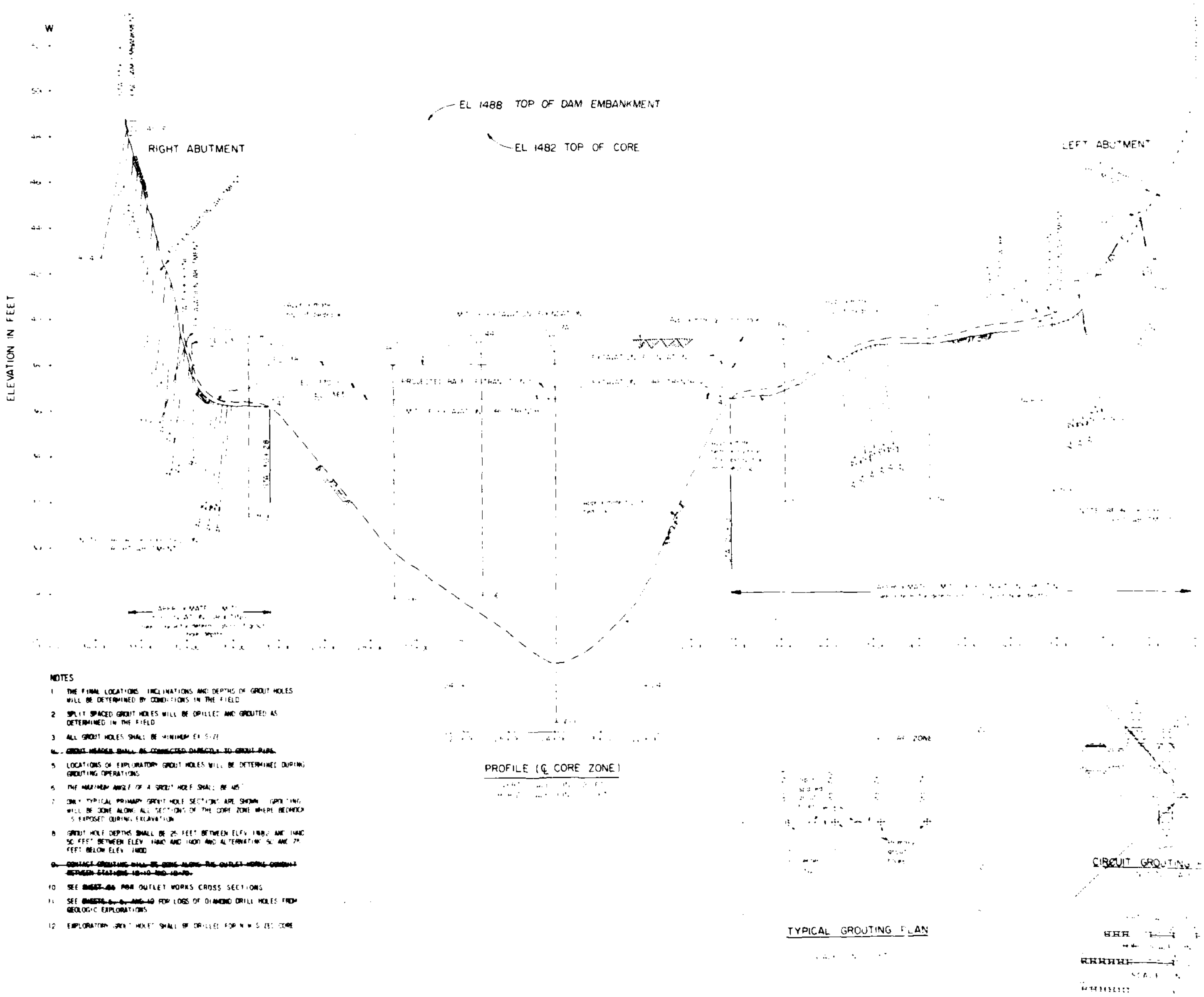
SCALE: 1 IN. = 10 FT.

10 0 10 20

GILA RIVER BASIN
PHOENIX AZ. AND VICINITY (INCL. NEW RIVER)

NEW RIVER DAM
DIKE NO. 1
FOUNDATION GEOLOGY
STA. 81+81 TO STA. 78+50

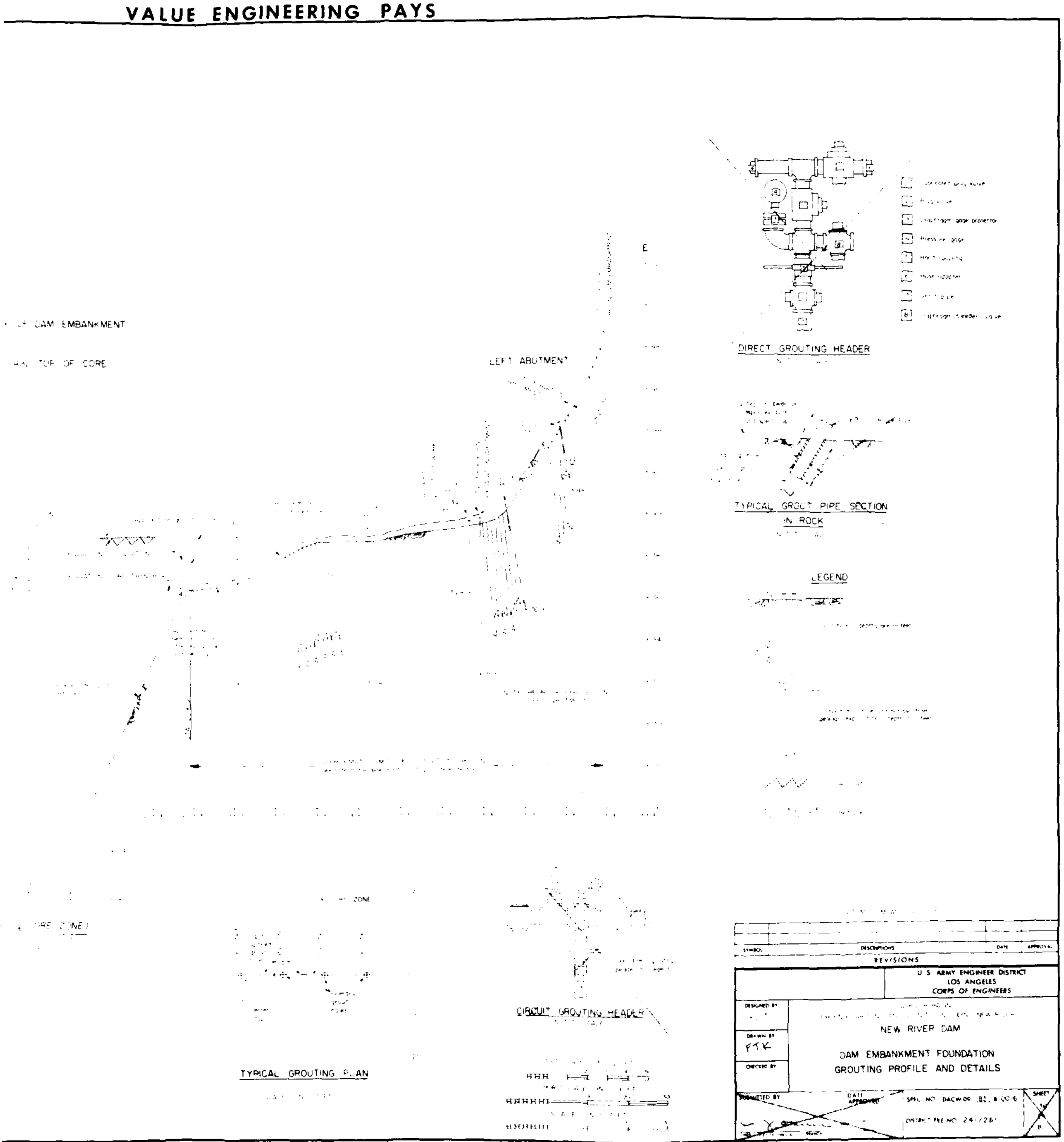
U.S. ARMY ENGINEER DISTRICT
LOS ANGELES CORPS OF ENGINEERS



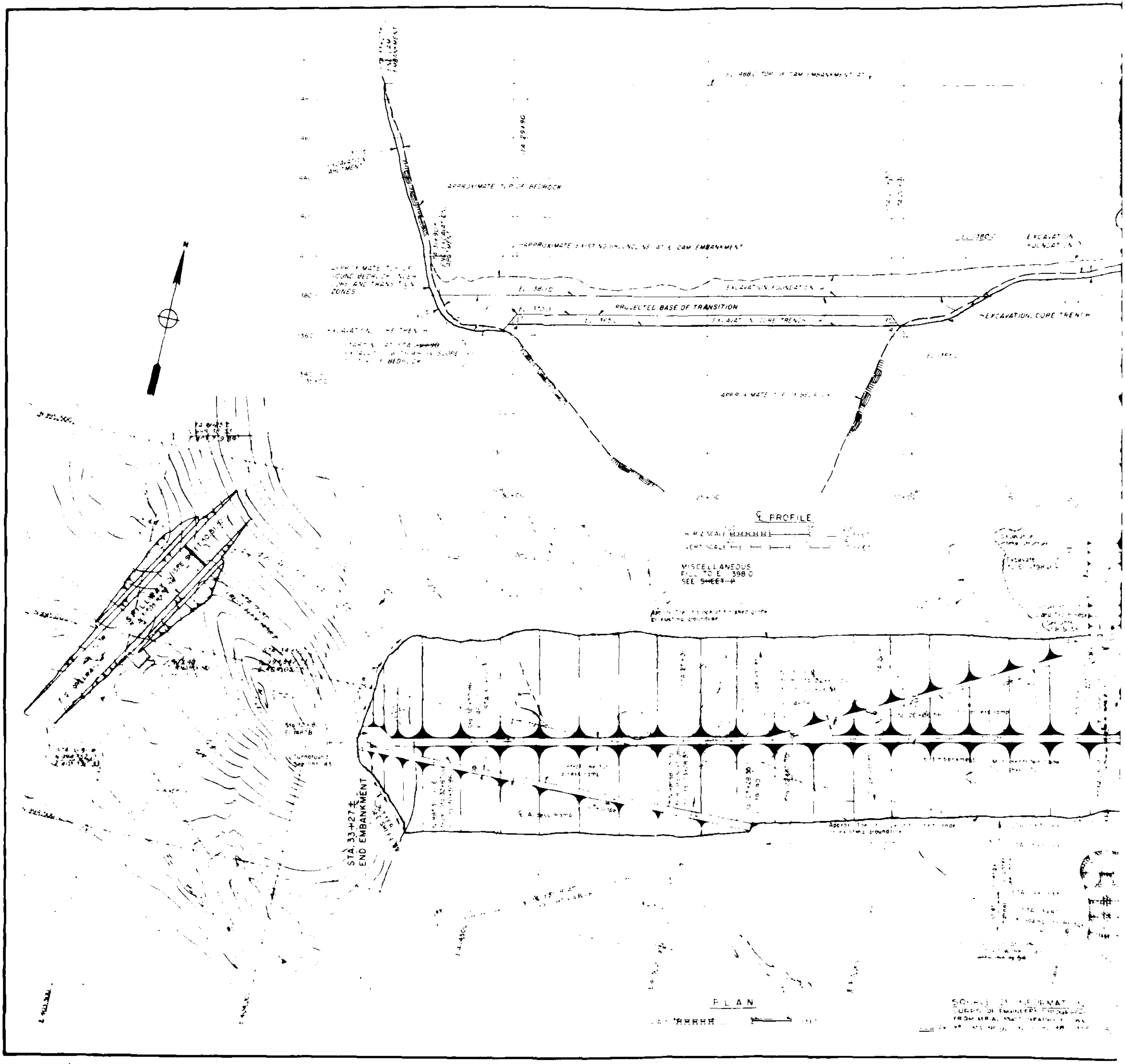
NOTES

1. THE FINAL LOCATIONS, INCLINATIONS AND DEPTHS OF GROUT HOLES WILL BE DETERMINED BY CONDITIONS IN THE FIELD
2. SPLIT SPACED GROUT HOLES WILL BE DRILLED AND GROUTED AS DETERMINED IN THE FIELD
3. ALL GROUT HOLES SHALL BE MINIMUM EX SIZE
4. ~~GROUT HEADER SHALL BE CONNECTED DIRECTLY TO GROUT PIPE.~~
5. LOCATIONS OF EXPLORATORY GROUT HOLES WILL BE DETERMINED DURING GROUTING OPERATIONS
6. THE MAXIMUM ANGLE OF A GROUT HOLE SHALL BE 45°
7. ONLY TYPICAL PRIMARY GROUT HOLE SECTIONS ARE SHOWN. GROUTING WILL BE DONE ALONG ALL SECTIONS OF THE CORE ZONE WHERE BEDROCK IS EXPOSED DURING EXCAVATION
8. GROUT HOLE DEPTHS SHALL BE 25 FEET BETWEEN ELEV 1482 AND 1440, 50 FEET BETWEEN ELEV 1440 AND 1400 AND ALTERNATING 50 AND 75 FEET BELOW ELEV 1400
9. ~~CONTACT GROUTING WILL BE DONE ALONG THE OUTLET WORKS CONDUIT BETWEEN STATIONS 10+10 AND 10+70.~~
10. SEE ~~[illegible]~~ FOR OUTLET WORKS CROSS SECTIONS
11. SEE ~~[illegible]~~ FOR LOGS OF DIAMOND DRILL HOLES FROM GEOLOGIC EXPLORATIONS
12. EXPLORATORY GROUT HOLES SHALL BE DRILLED FOR N.W. SIZE CORE

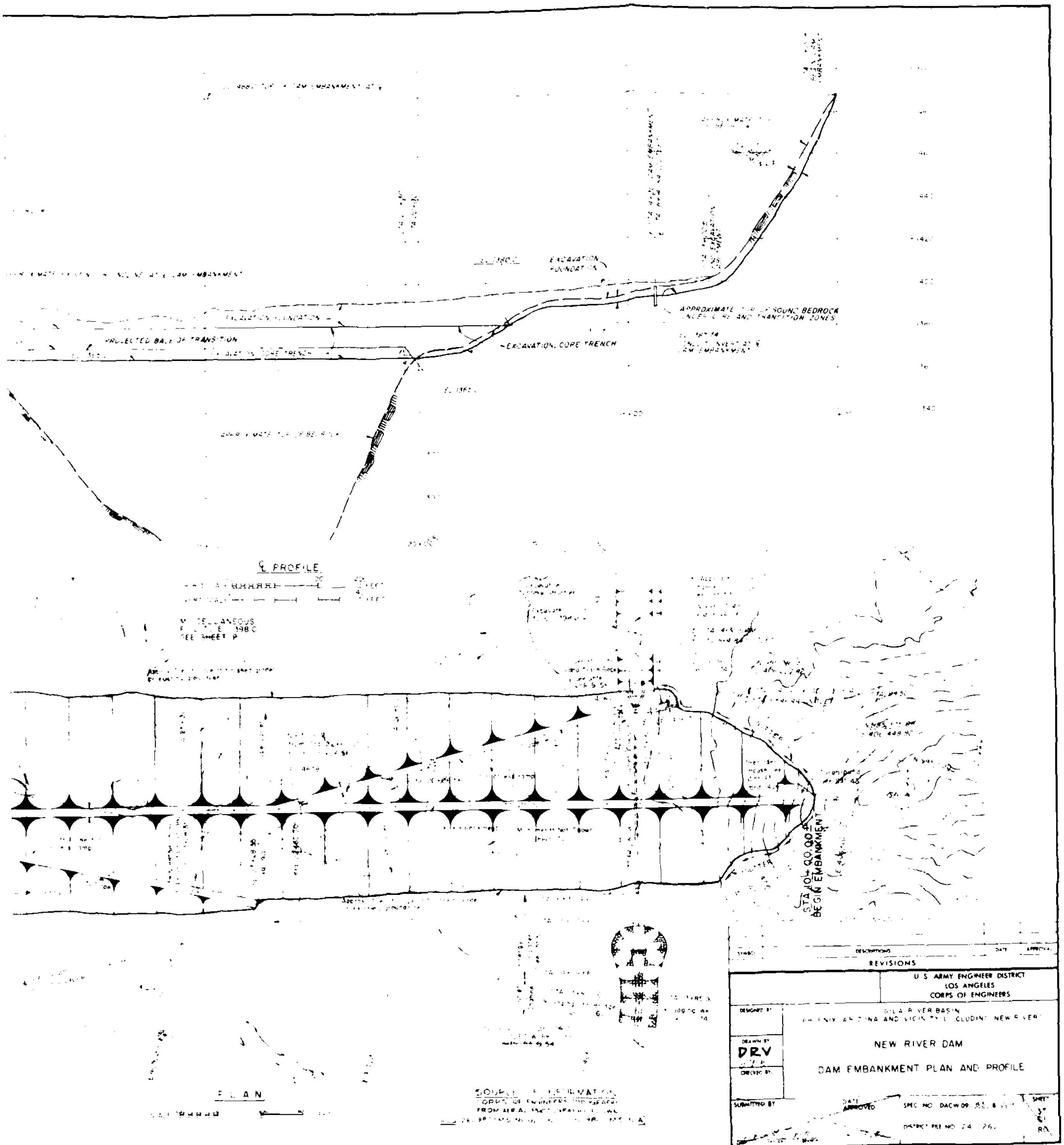
VALUE ENGINEERING PAYS



SAFETY PAYS

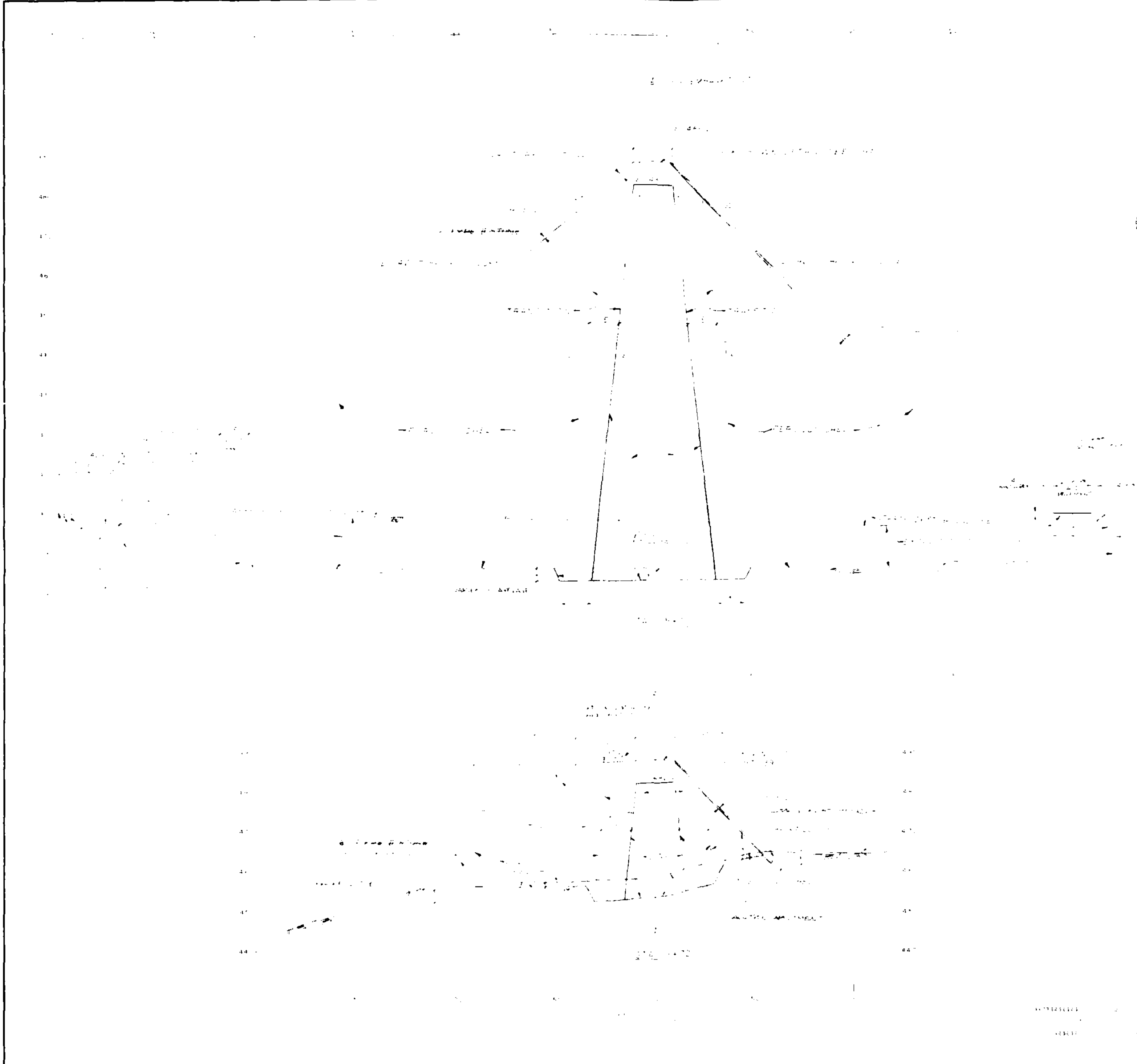


APPROXIMATE TOP OF BEDROCK
APPROXIMATE EXISTING GROUND LINE AT ℄ DAM EMBANKMENT
PROJECTED BASE OF TRANSITION
EXCAVATION, CORE TRENCH
℄ PROFILE
MISCELLANEOUS FILL TO EL 398.0
STA. 33+27± END EMBANKMENT
PLAN



EXCAVATION FOUNDATION
APPROXIMATE TOP OF SOUND BEDROCK
EXCAVATION, CORE TRENCH
PROJECTED BASE OF TRANSITION
℄ PROFILE
MISCELLANEOUS
SEE SHEET
STA 104 00.00
BEGIN EMBANKMENT
PLAN
REVISIONS
SYMBOL
DESCRIPTIONS
DATE
APPROVAL
U S ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS
DESIGNED BY
DRAWN BY
DRV
CHECKED BY
SUBMITTED BY
NEW RIVER DAM
DAM EMBANKMENT PLAN AND PROFILE
DATE
APPROVED
SPEC NO DACW09
DISTRICT FILE NO
SHEET

VALUE ENGINEERING PAYS

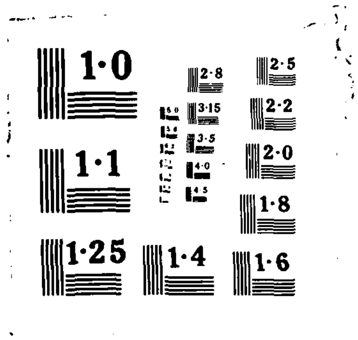


SAFETY PAYS

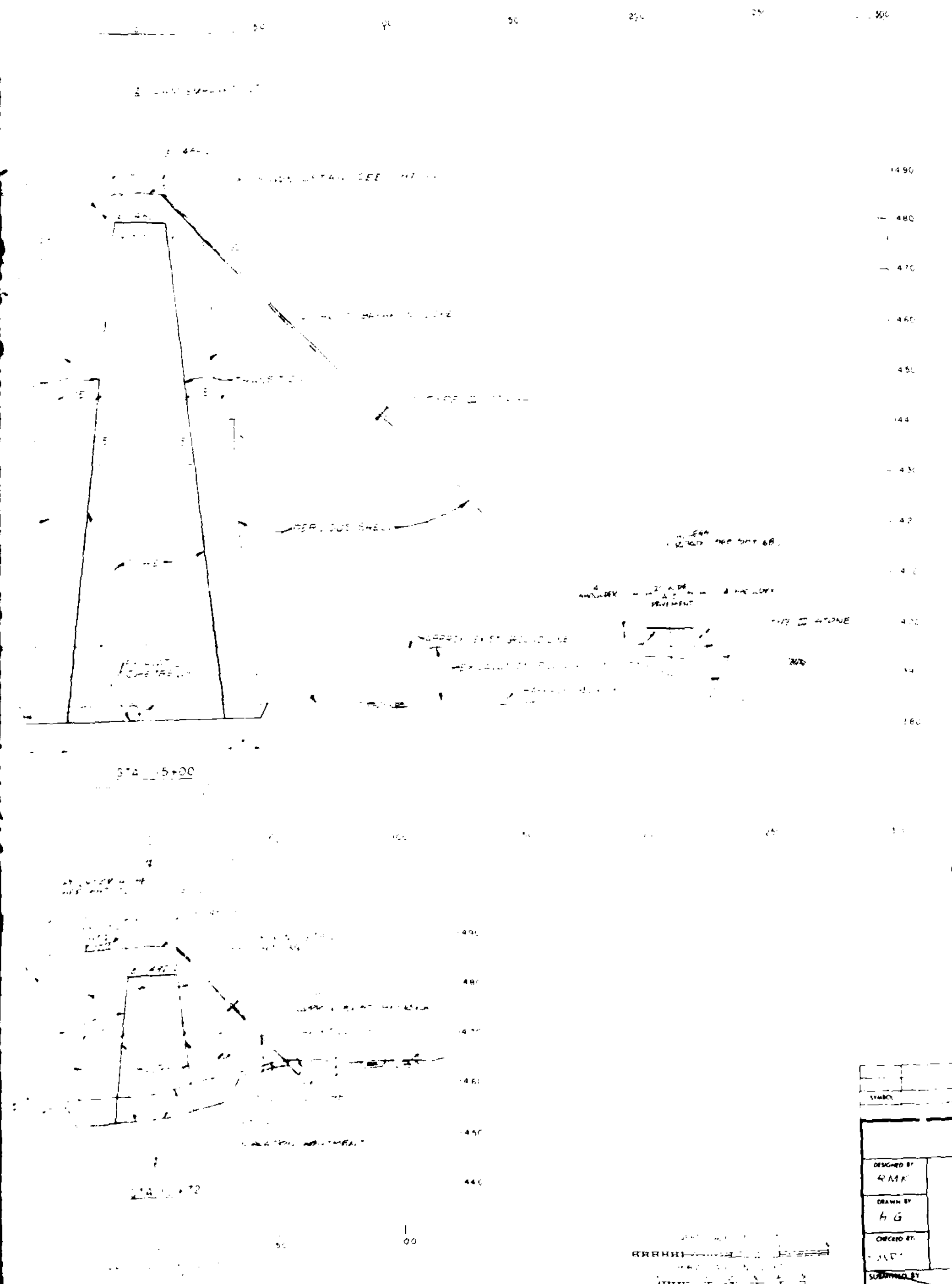
AD-A168 748 NEW RIVER DAM FOUNDATION REPORT GILA RIVER BASIN: PHOENIX ARIZONA AND VICINITY (INCLUDING NEW RIVER) (U) ARMY ENGINEER DISTRICT LOS ANGELES CA OCT 85 4/4

UNCLASSIFIED F/G 13/2 NL

END
DATE
FILMED
8-86


1·0
1·1
1·25
1·4
1·6
1·8
2·0
2·2
2·5
2·8
3·15
3·5
4·0
4·5

VALUE ENGINEERING PAYS



NOTE: VIEWS OF CROSS-SECTIONS ARE LOOKING TOWARD DECREASING STATIONS

DATUM IS MEAN SEA LEVEL

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

GILA RIVER BASIN
PHOENIX, ARIZONA AND VICINITY (INCLUDING NEW RIVER)

NEW RIVER DAM

DAM EMBANKMENT
CROSS SECTIONS

DESIGNED BY: RMK

DRAWN BY: H G

CHECKED BY:

SUBMITTED BY

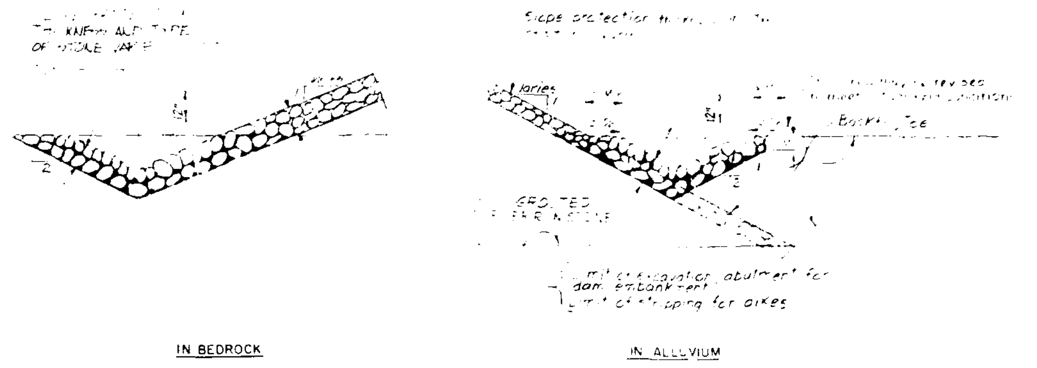
DATE APPROVED

SPEC NO. DACW09-83-B-0016

DISTRICT FILE NO. 24-263

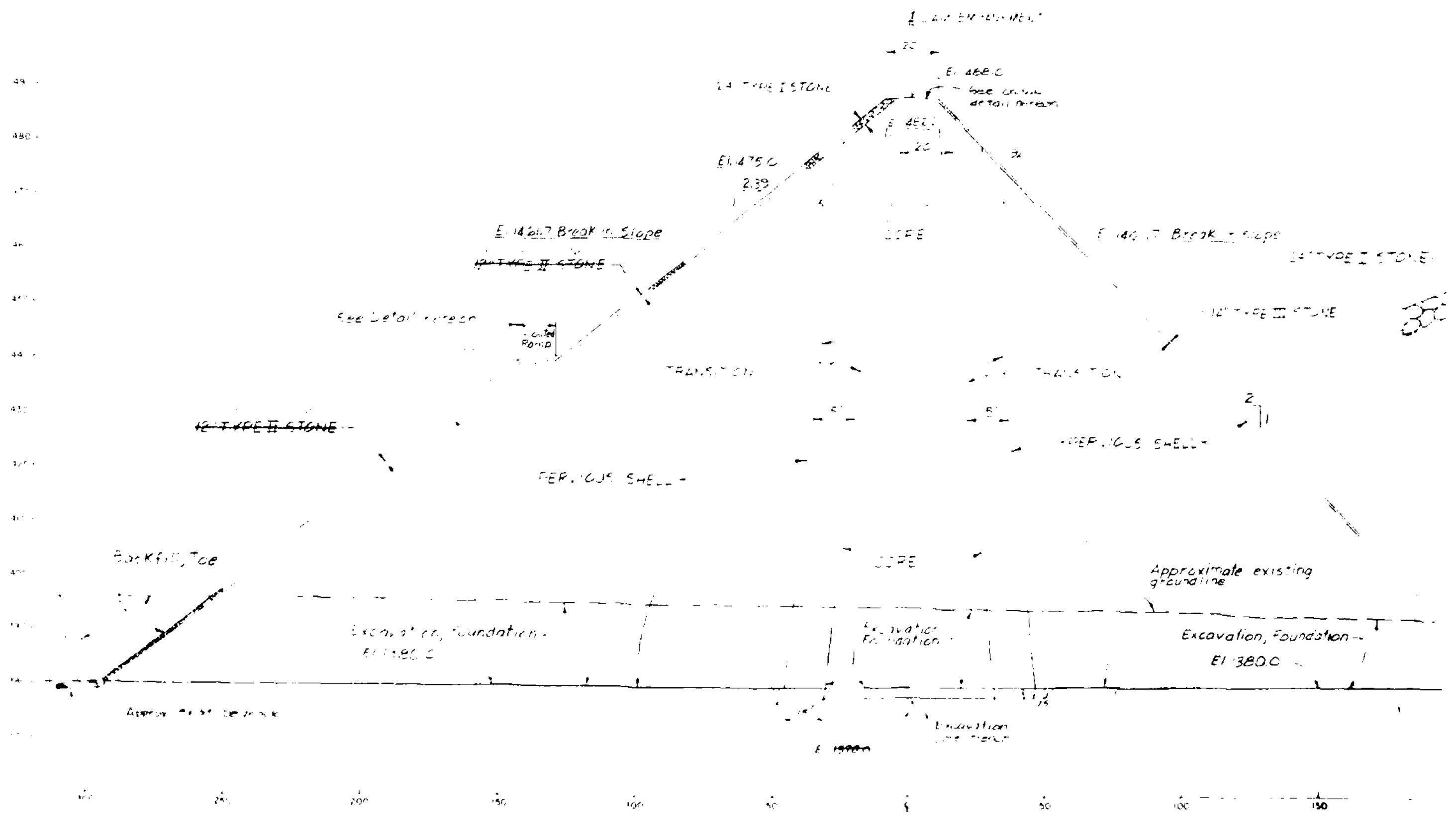
SHEET 80

SAFETY PAYS



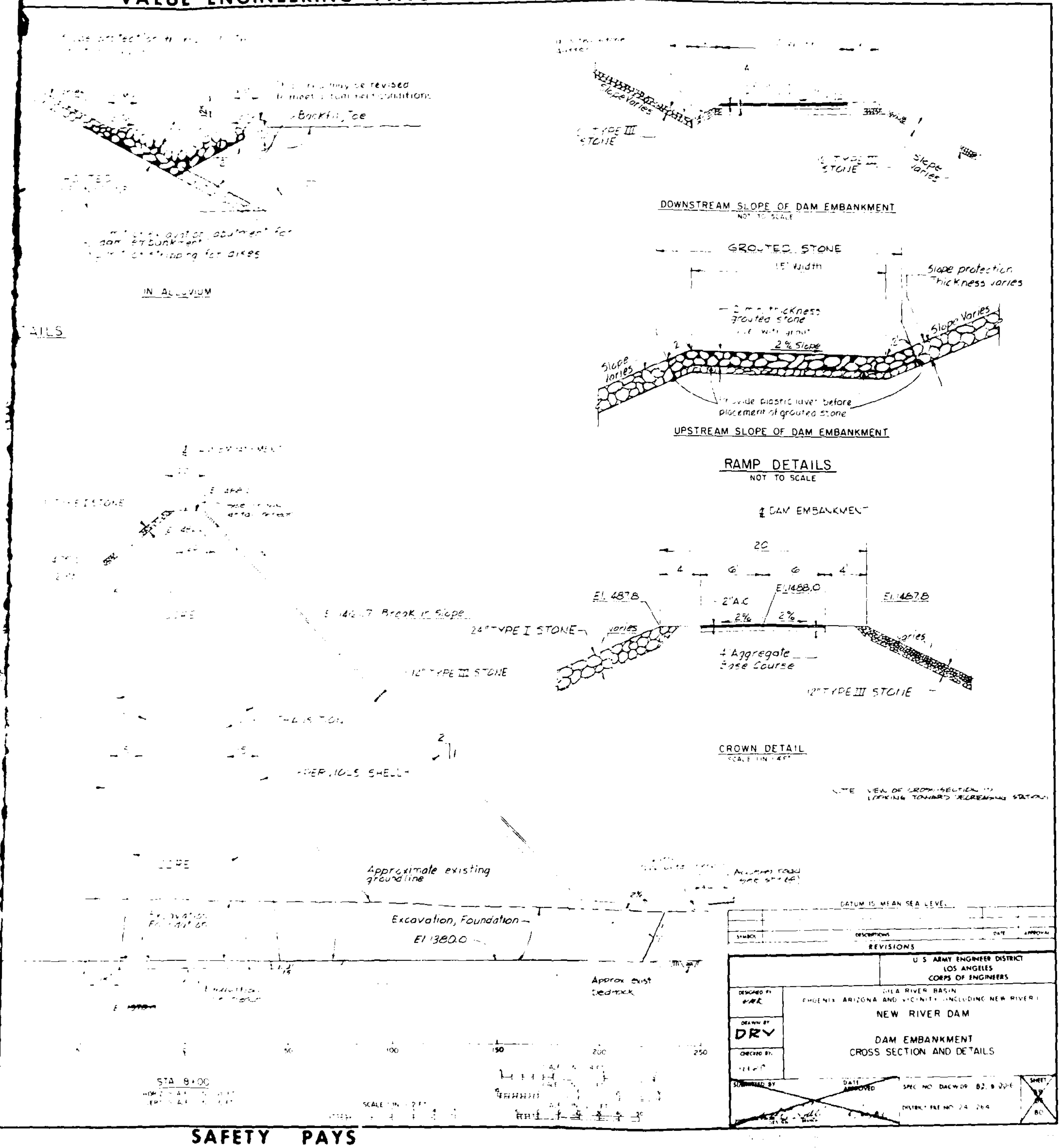
GUTTER DETAILS

SCALE 1 IN = 2 FT



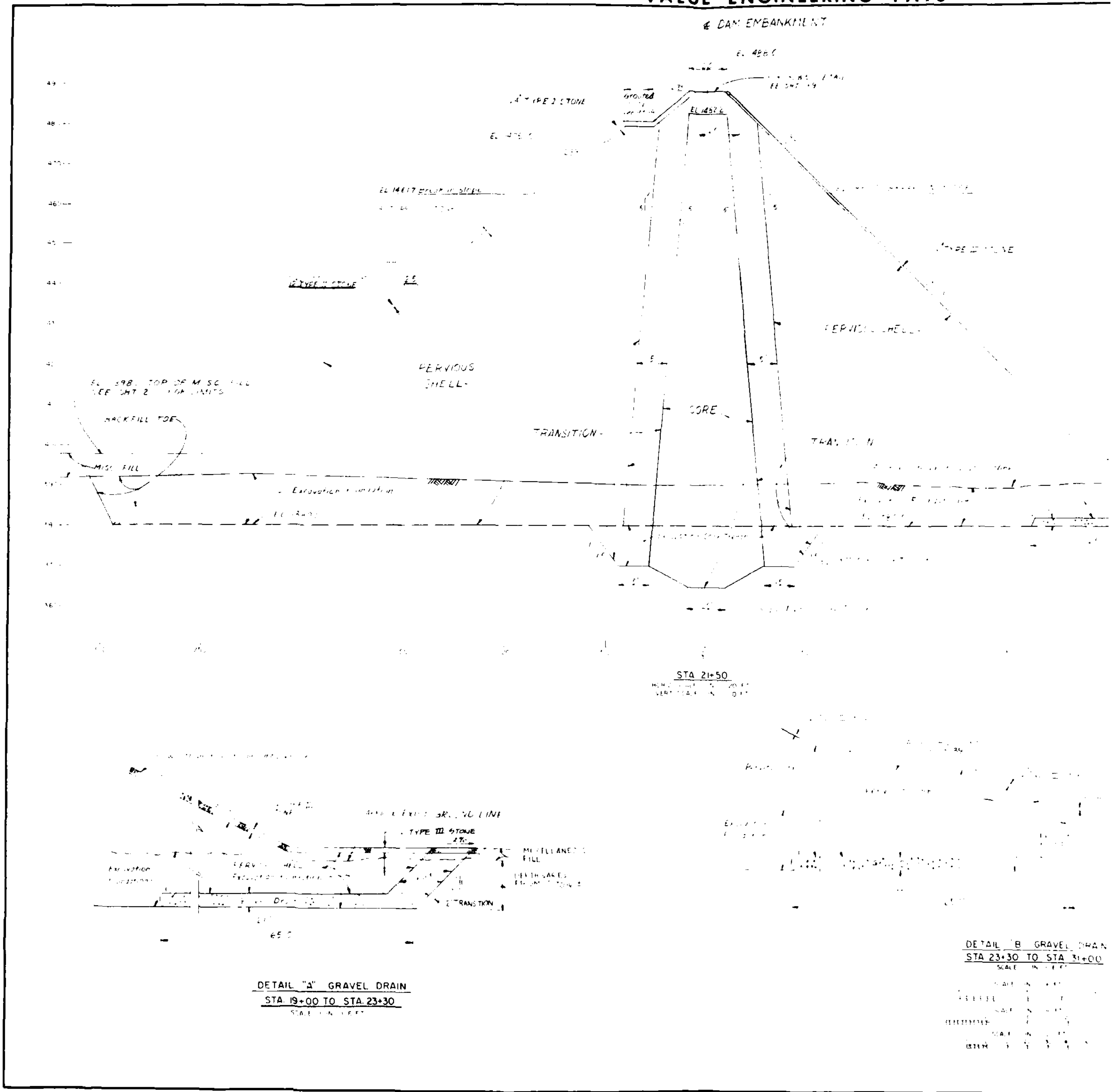
STA 18+00

VALUE ENGINEERING PAYS



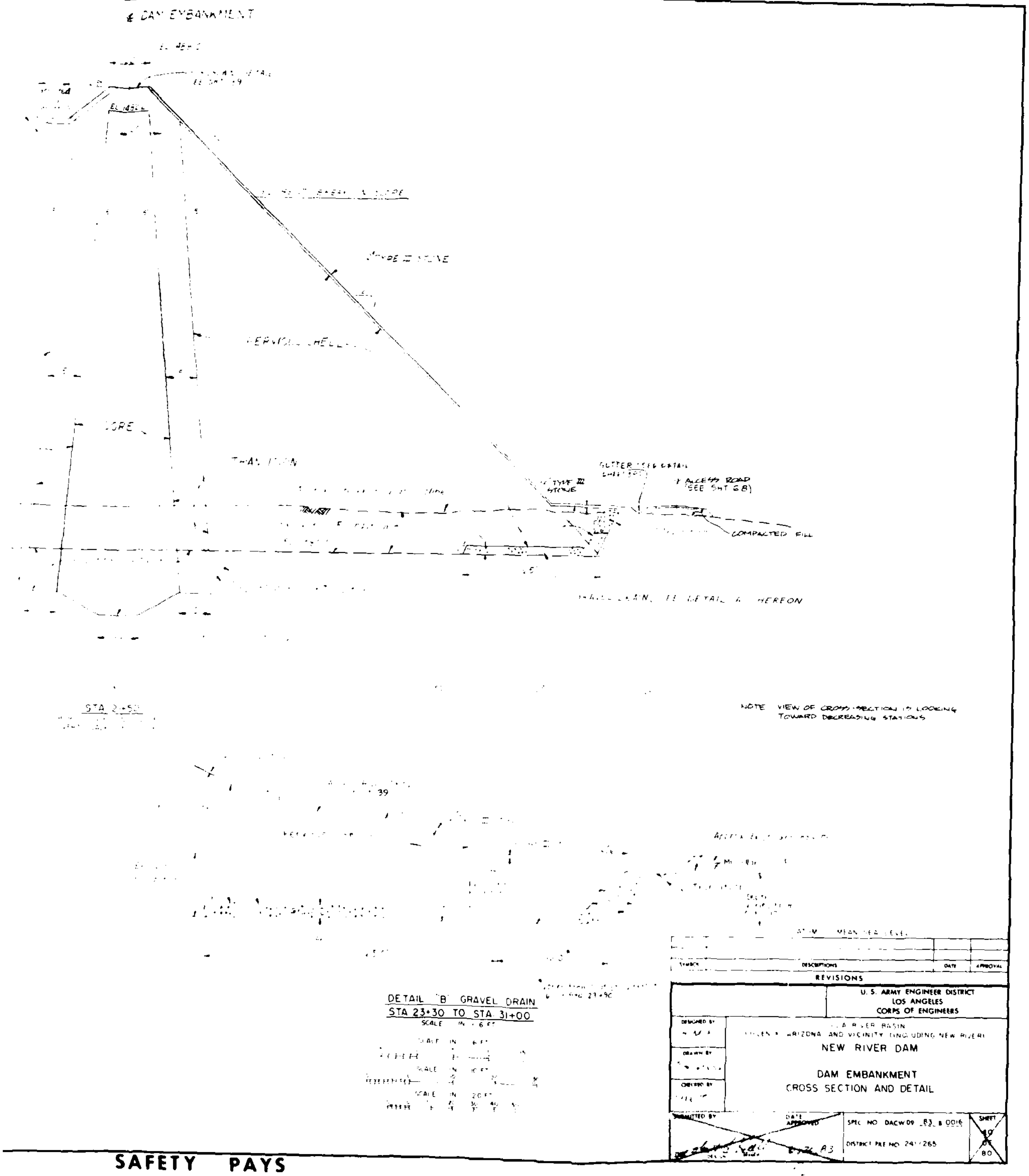
SAFETY PAYS

VALUE ENGINEERING PAYS

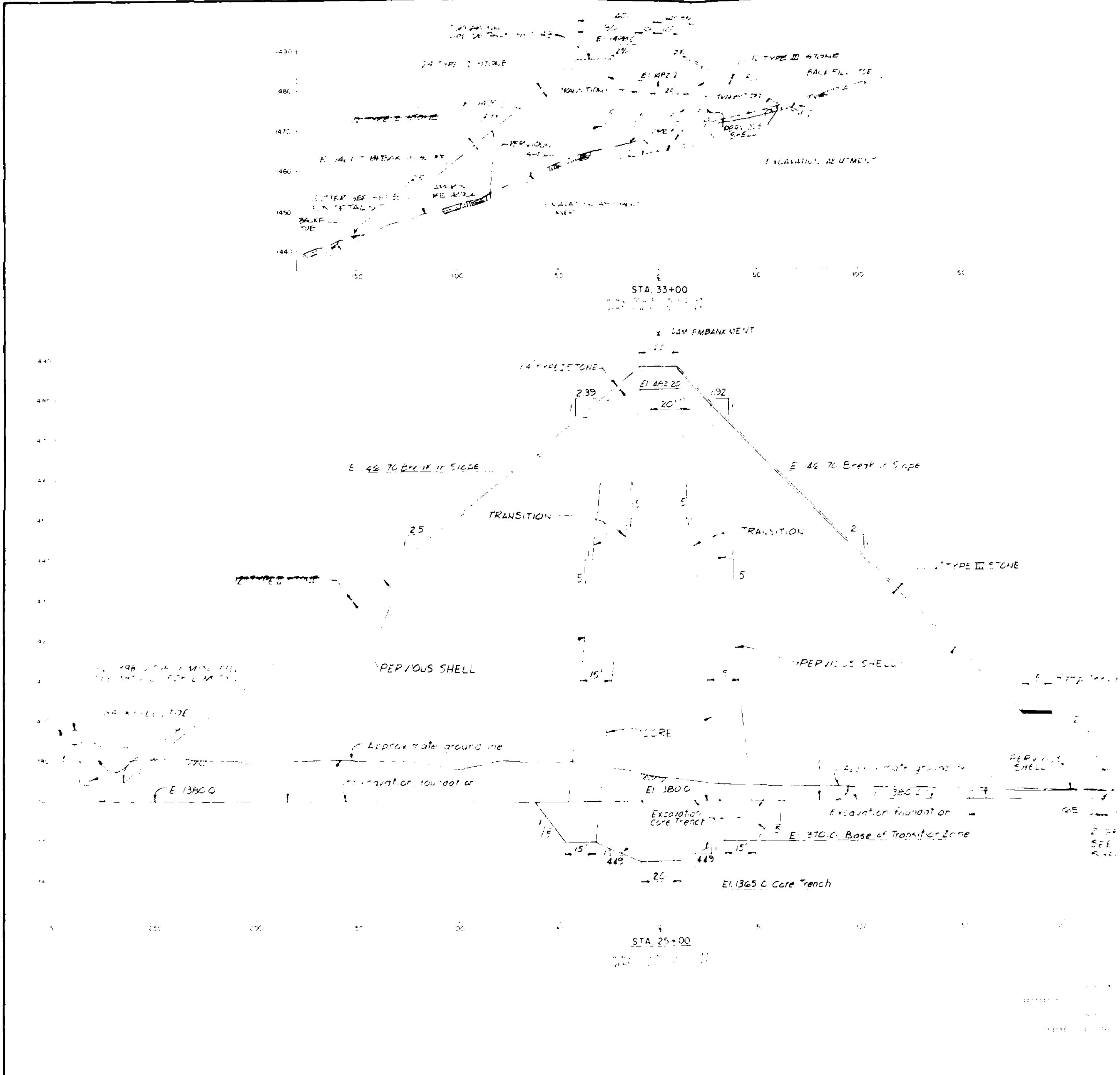


SAFETY PAYS

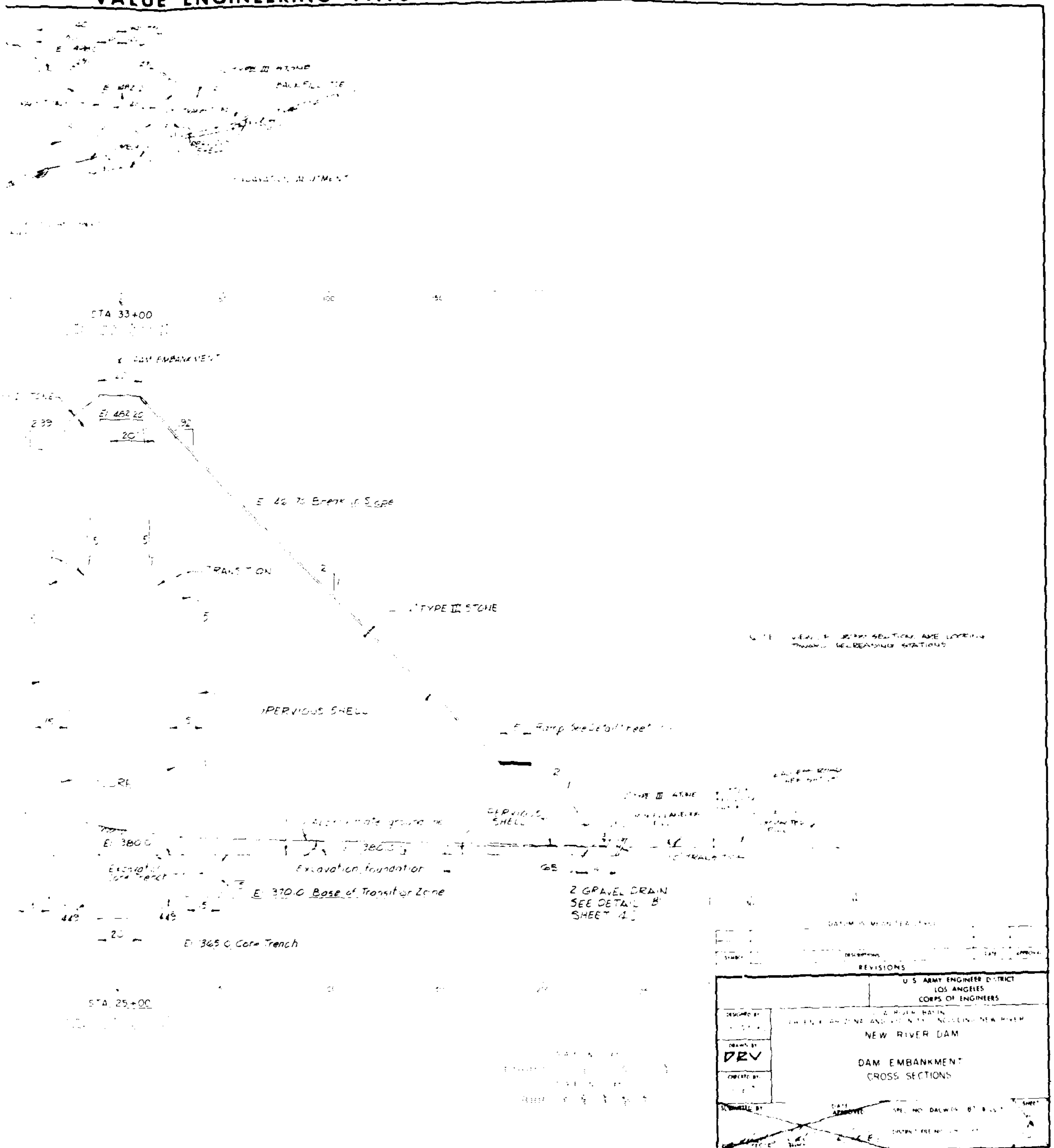
VALUE ENGINEERING PAYS



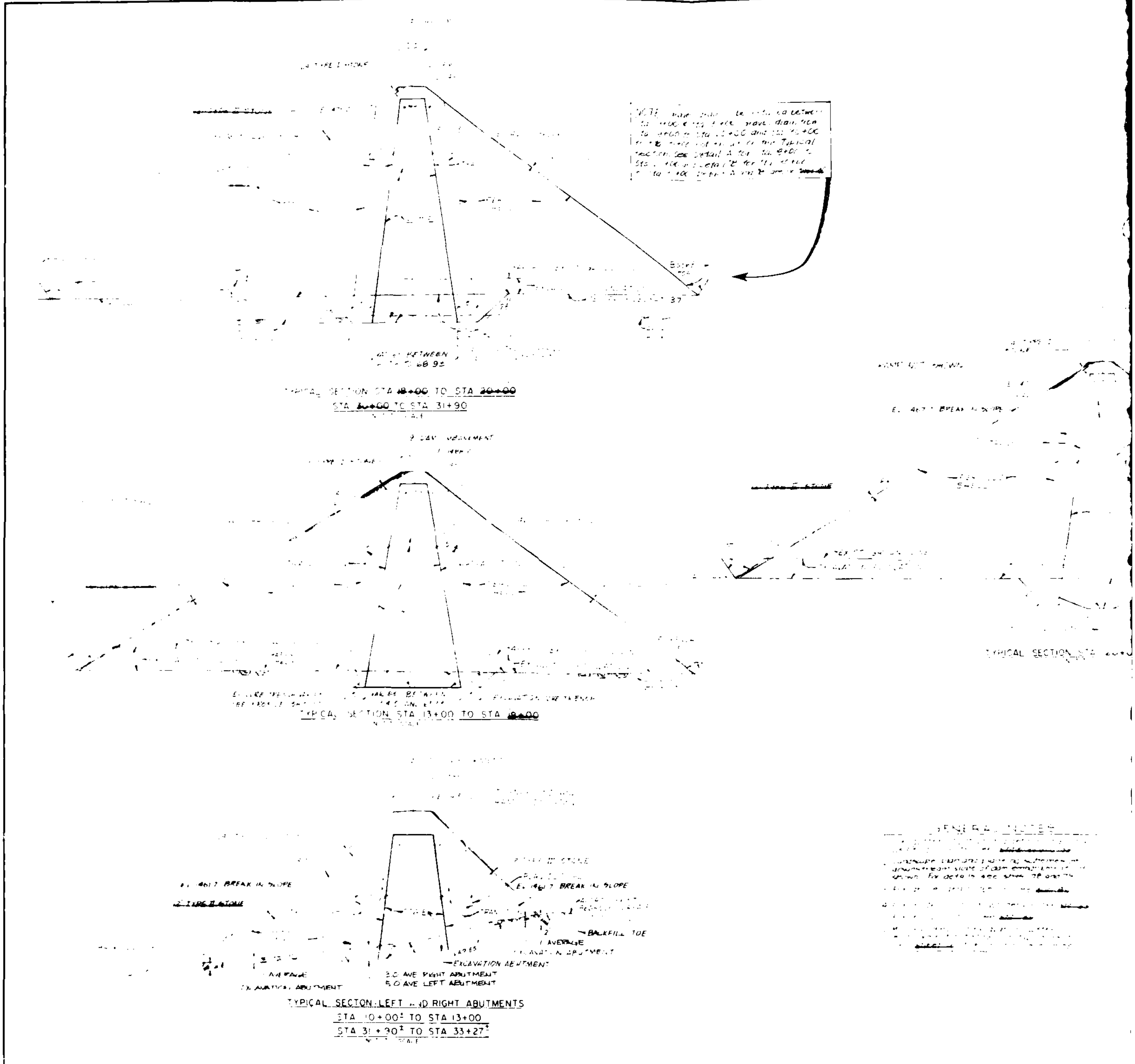
SAFETY PAYS


STA. 33+00
DAM EMBANKMENT
El 1482.20
2.39
1.92
El 1446.76 Break in Slope
TRANSITION
PERVIOUS SHELL
CORE
Approximate ground line
El 1380.0
Excavation Core Trench
El 1370.0 Base of Transition Zone
El 1365.0 Core Trench
STA. 25+00

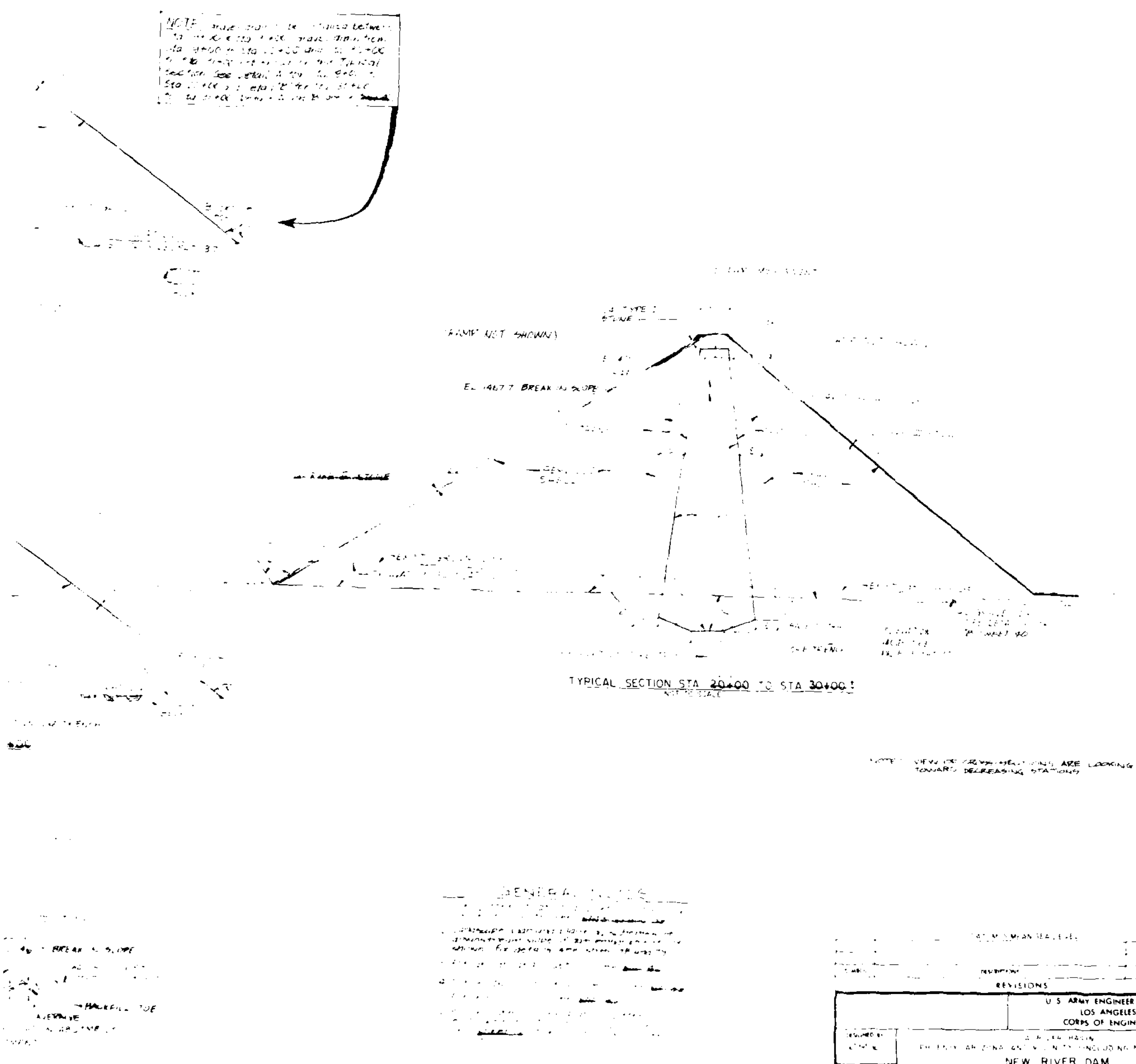
VALUE ENGINEERING PAYS



SAFETY PAYS


TYPICAL SECTION STA 18+00 TO STA 20+00
STA 31+90
TYPICAL SECTION STA 13+00 TO STA 18+00
EL 4617 BREAK IN SLOPE
BACKFILL TOE
AVERAGE
EXCAVATION ABUTMENT
3.0 AVE RIGHT ABUTMENT
5.0 AVE LEFT ABUTMENT
TYPICAL SECTION LEFT AND RIGHT ABUTMENTS
STA 10+00± TO STA 13+00
STA 31+90± TO STA 33+27±
TYPICAL SECTION STA

VALUE ENGINEERING PAYS



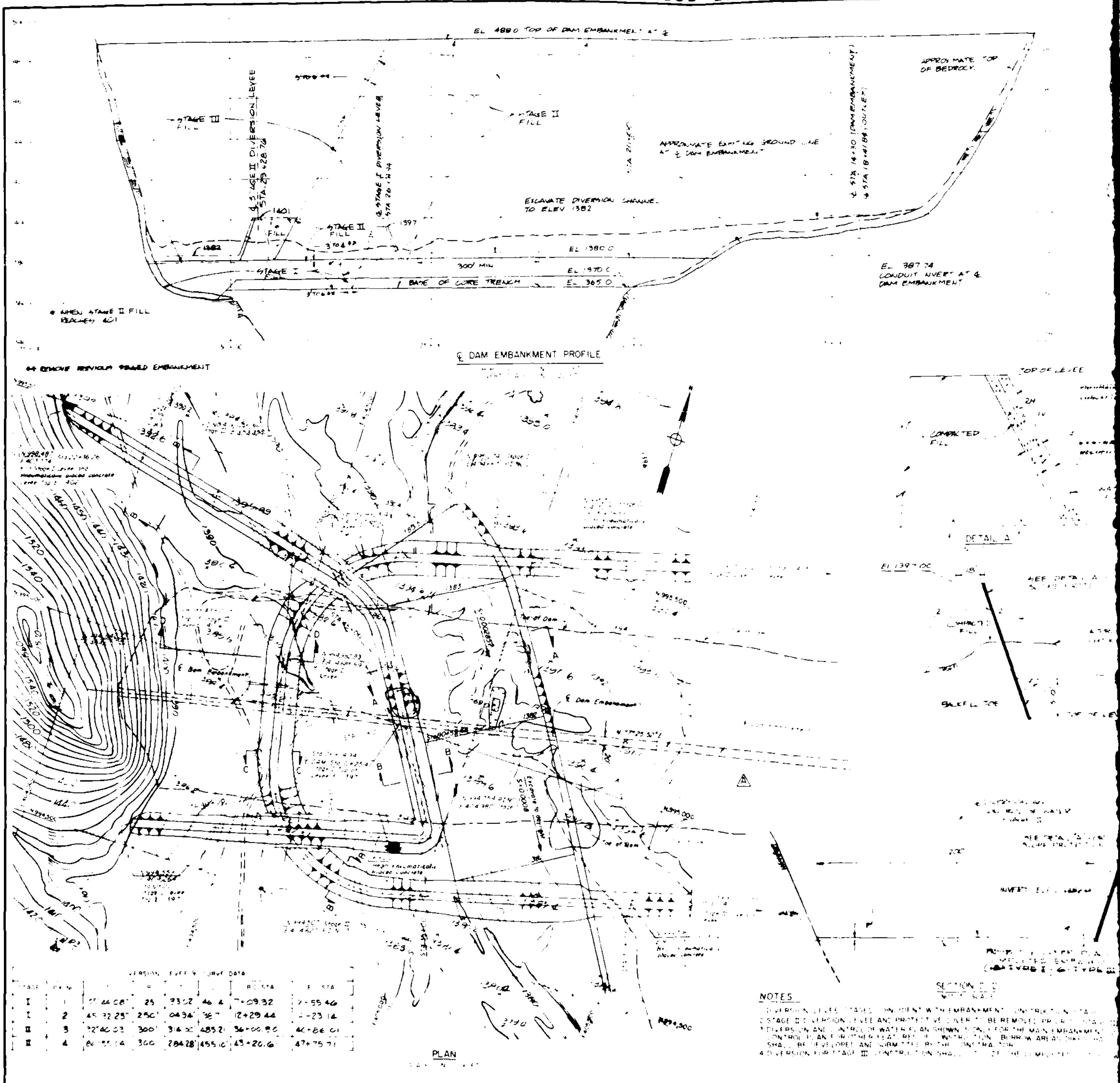
REVISIONS

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

NEW RIVER DAM

DAM EMBANKMENT
TYPICAL SECTIONS

SAFETY PAYS

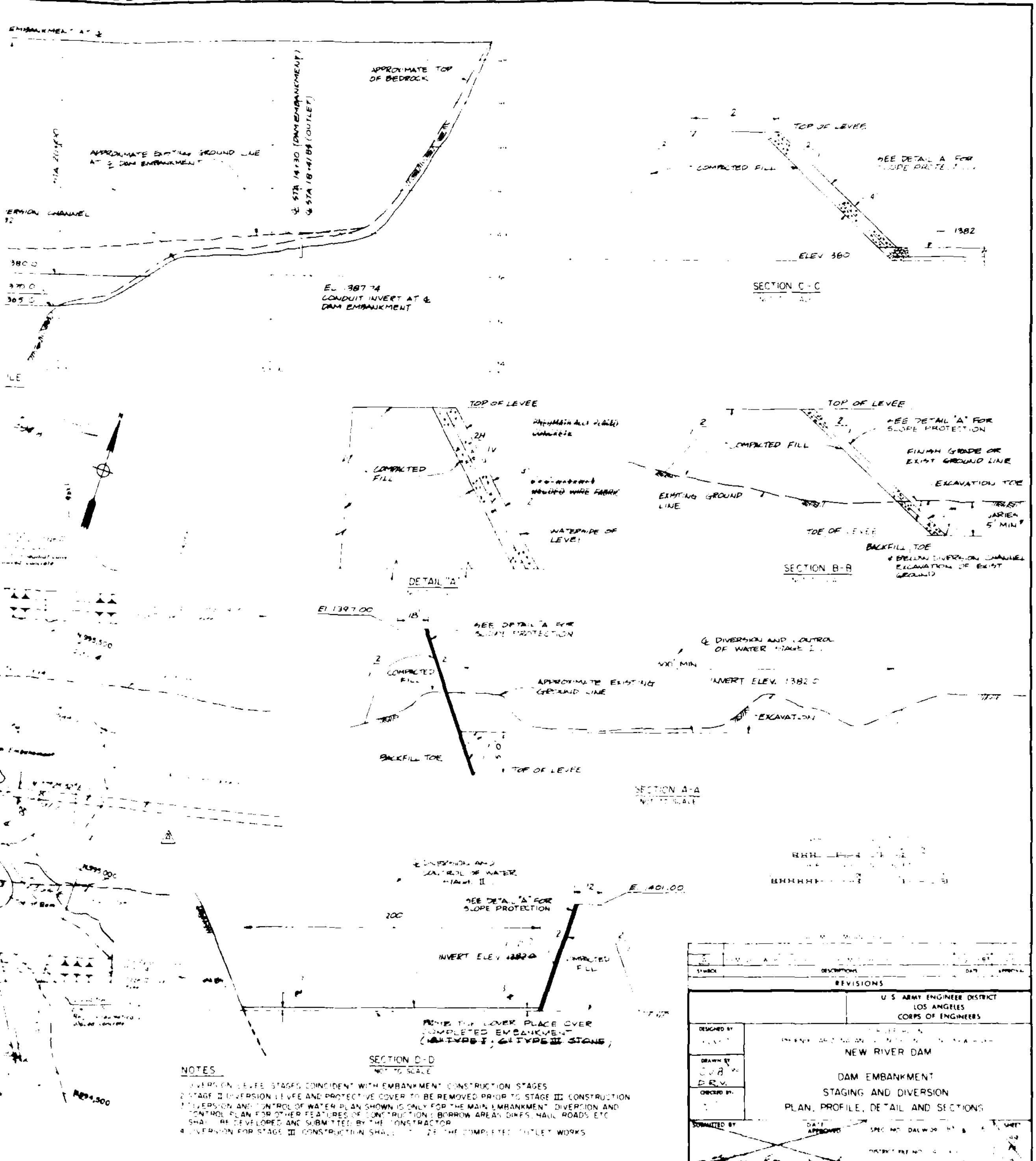


STAGE	CURVE					BC STA	EC STA
I	1	[illegible]°44'08"	25	33.02	46.4	7+09.32	7+55.46
I	2	45°32'23"	250'	04.94'	387	12+29.44	[illegible]+23.14
II	3	32°40'03"	300'	314.50	485.2	36+00.86	40+86.01
II	4	84°55'24"	300	284.28'	455.10'	43+20.6	47+75.71

NOTES

1. DIVERSION LEVEE STAGES CONCURRENT WITH EMBANKMENT CONSTRUCTION STAGES
2. STAGE II DIVERSION LEVEE AND PROTECTIVE COVER TO BE REMOVED PRIOR TO STAGE III
3. DIVERSION AND CONTROL OF WATER PLAN SHOWN IS ONLY FOR THE MAIN EMBANKMENT. CONTROL PLAN FOR OTHER FEATURES OF CONSTRUCTION, BORROW AREAS [illegible] SHALL BE DEVELOPED AND SUBMITTED BY THE CONTRACTOR.
4. DIVERSION FOR STAGE III CONSTRUCTION SHALL [illegible] THE COMPLETED [illegible]

VALUE ENGINEERING PAYS



NOTES
...VERSION LEVEE STAGES COINCIDENT WITH EMBANKMENT CONSTRUCTION STAGES
2. STAGE II DIVERSION LEVEE AND PROTECTIVE COVER TO BE REMOVED PRIOR TO STAGE III CONSTRUCTION
3. DIVERSION AND CONTROL OF WATER PLAN SHOWN IS ONLY FOR THE MAIN EMBANKMENT. DIVERSION AND CONTROL PLAN FOR OTHER FEATURES OF CONSTRUCTION (BORROW AREAS, DIKES, HAUL ROADS, ETC.) SHALL BE DEVELOPED AND SUBMITTED BY THE CONSTRUCTOR.
4. DIVERSION FOR STAGE III CONSTRUCTION SHALL ... THE COMPLETED OUTLET WORKS

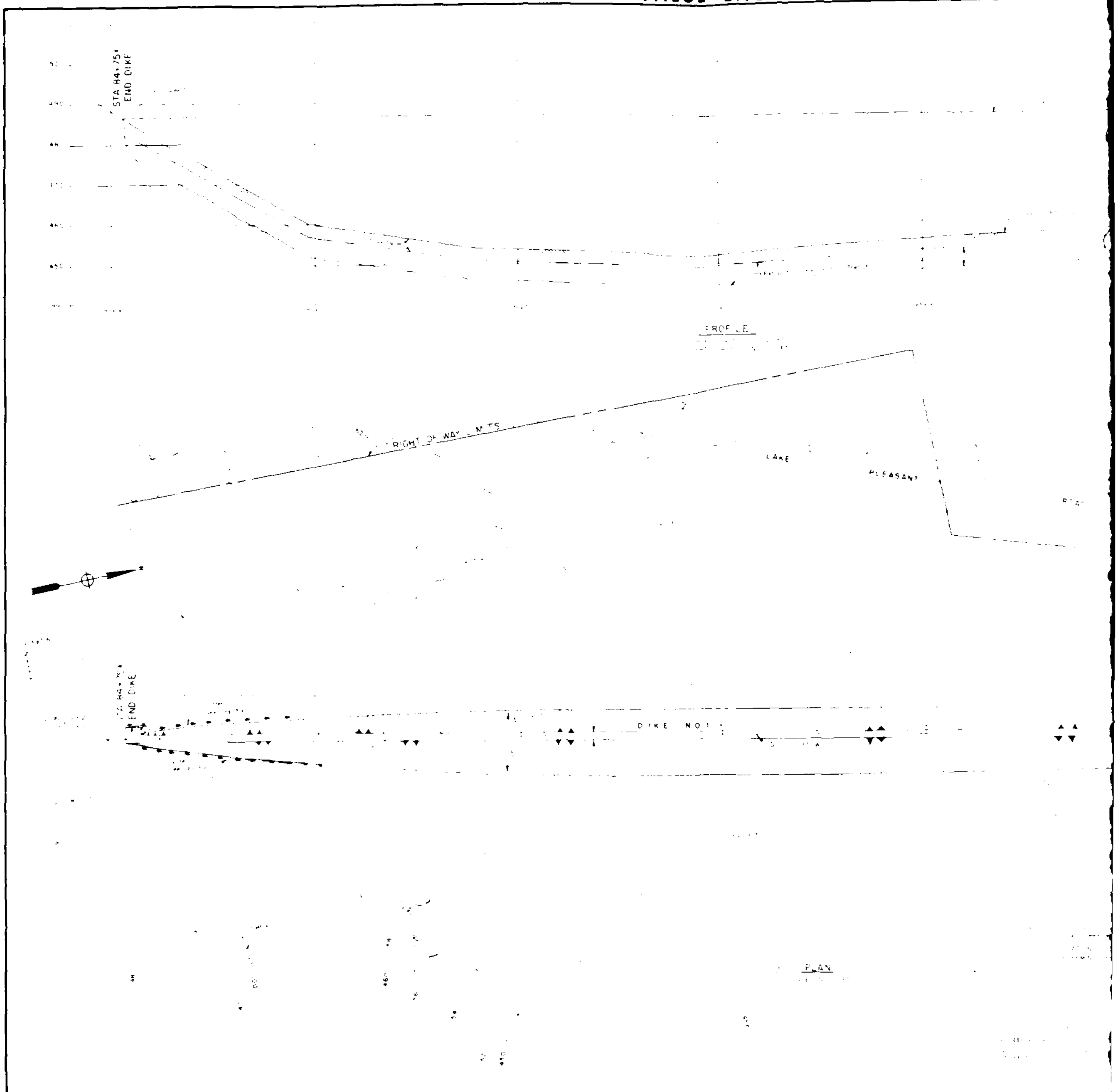
REVISIONS

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

NEW RIVER DAM
DAM EMBANKMENT
STAGING AND DIVERSION
PLAN, PROFILE, DETAIL AND SECTIONS

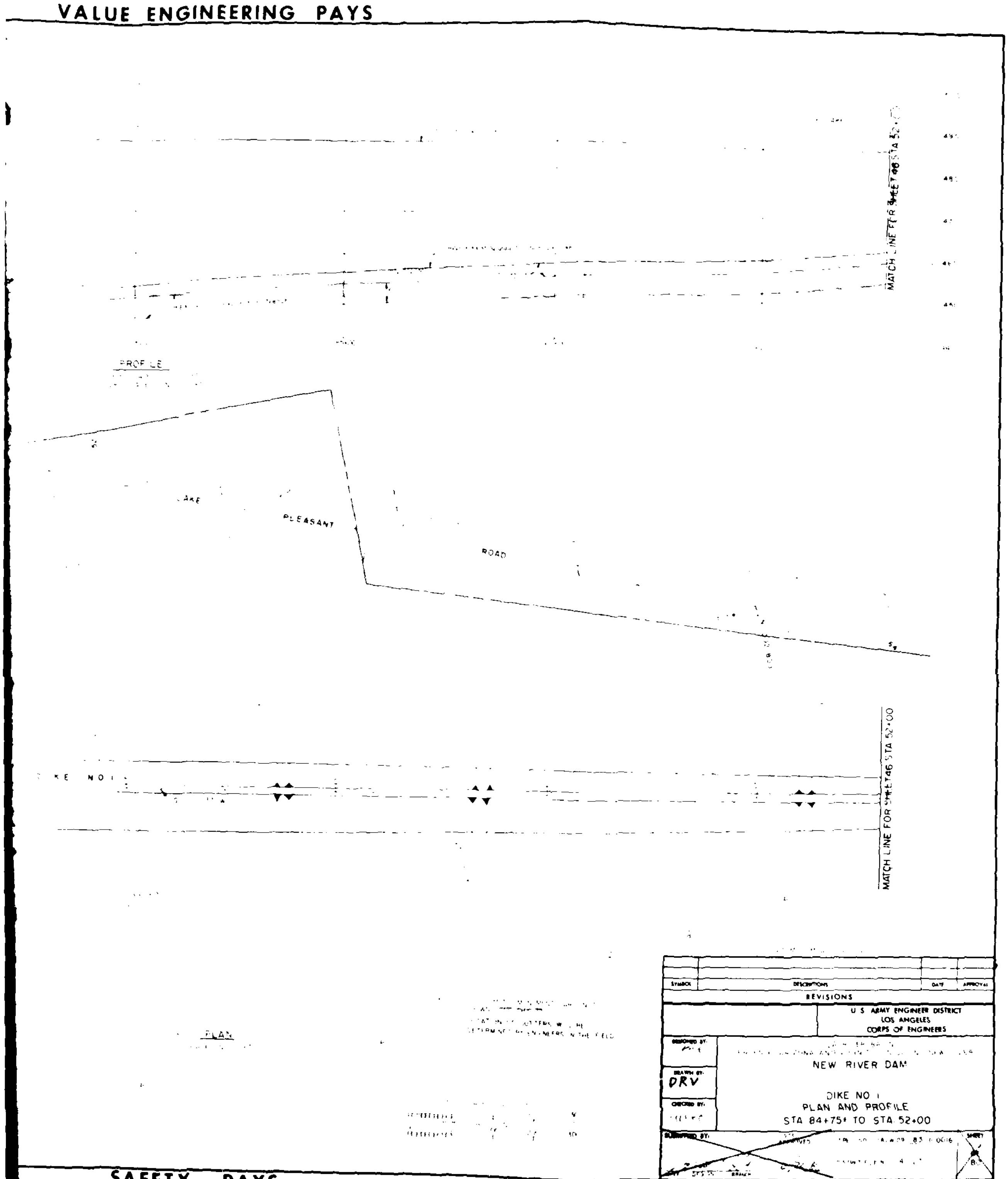
SAFETY PAYS

VALUE ENGINEERING PAYS



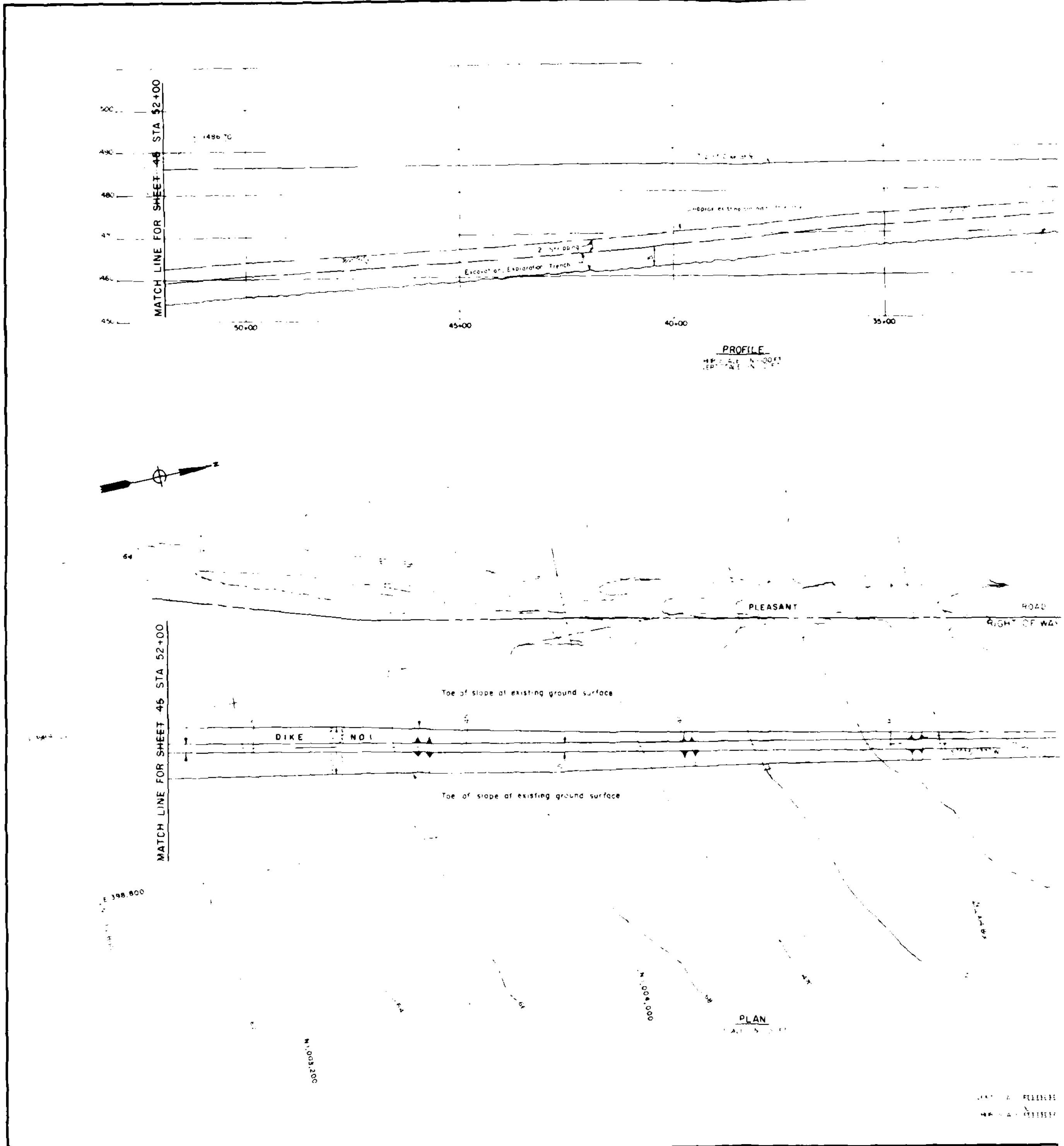
SAFETY PAYS

VALUE ENGINEERING PAYS



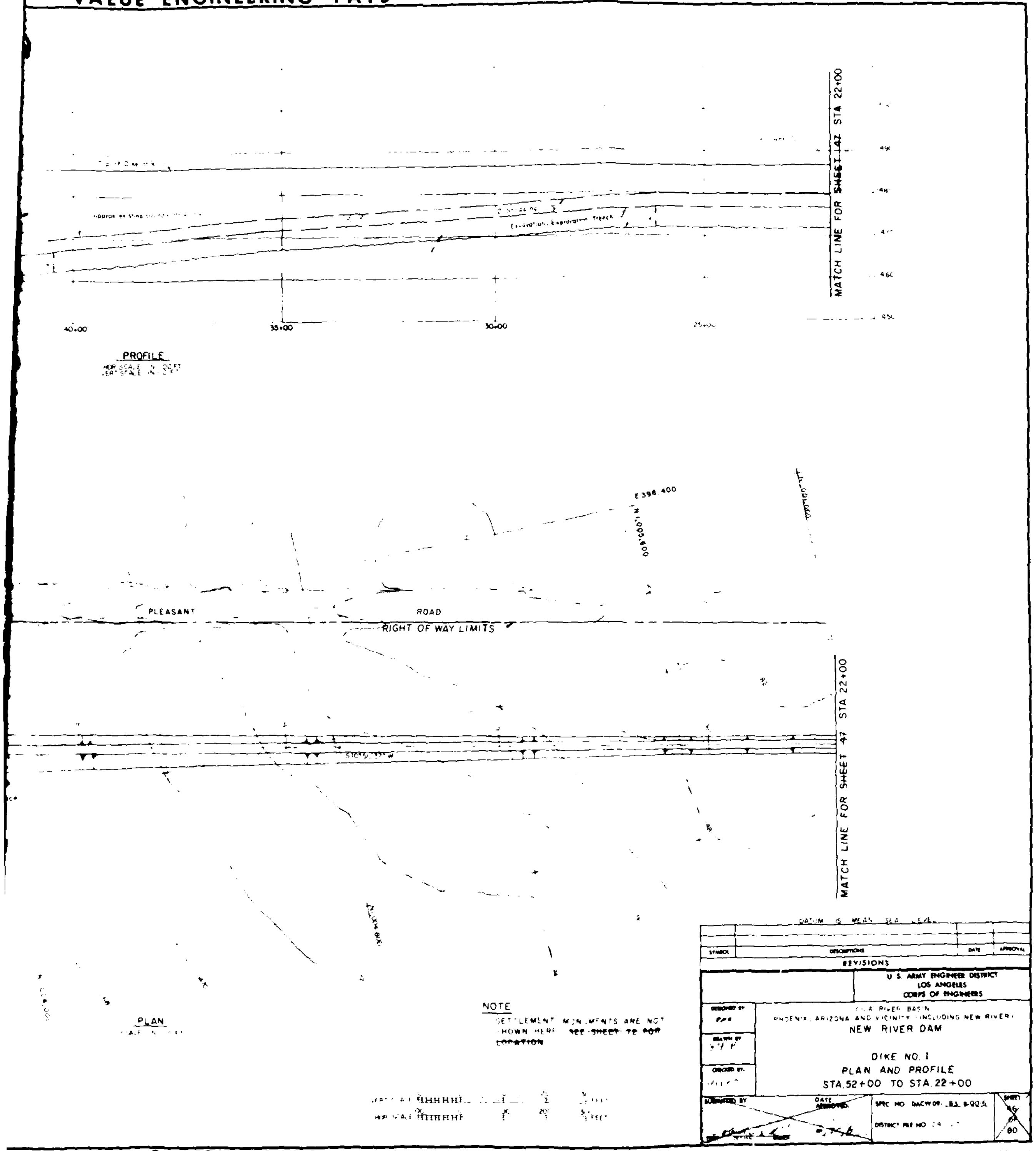
SAFETY PAYS

VALUE ENGINEERING PAYS



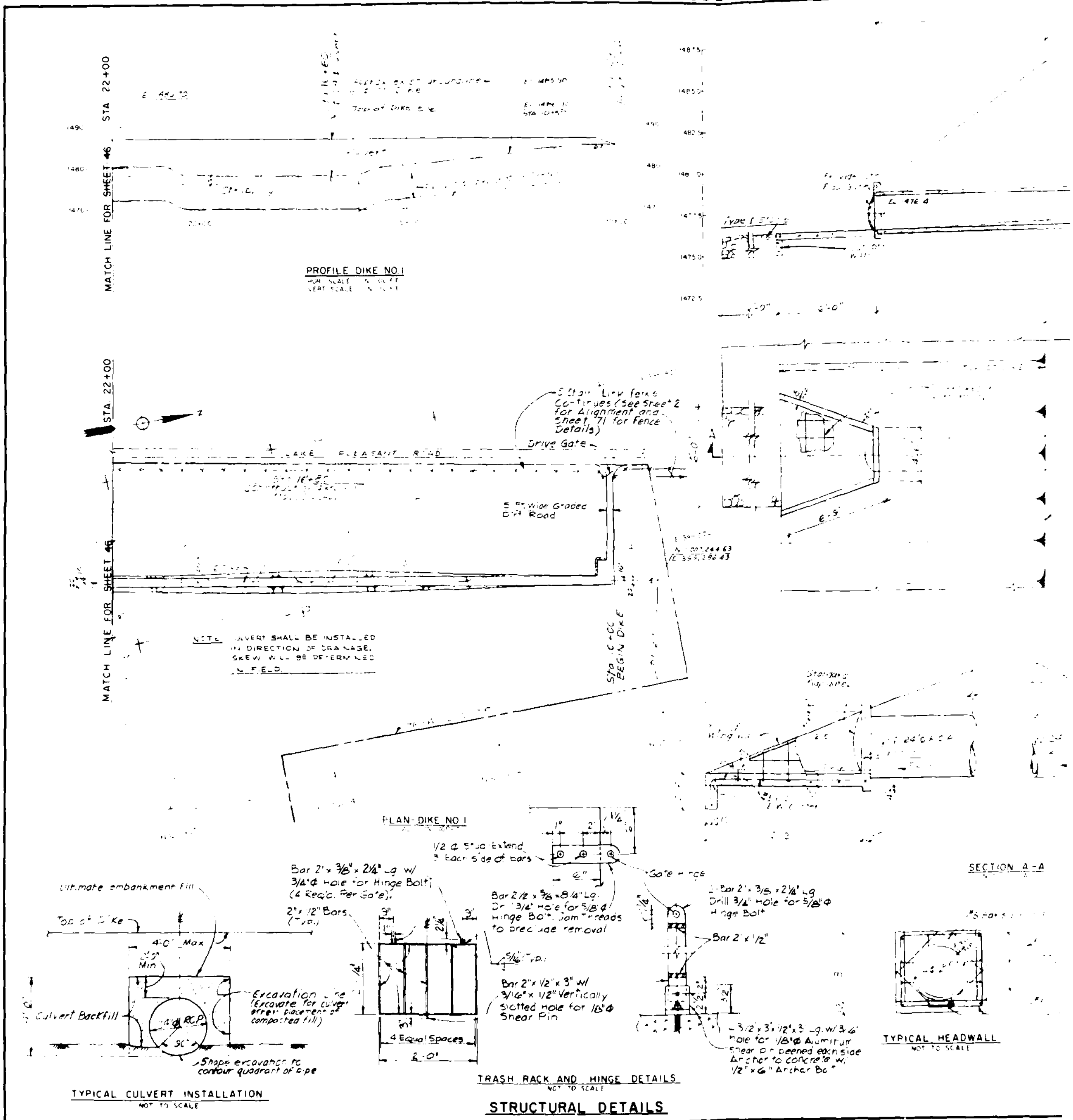
SAFETY PAYS

VALUE ENGINEERING PAYS



SAFETY PAYS

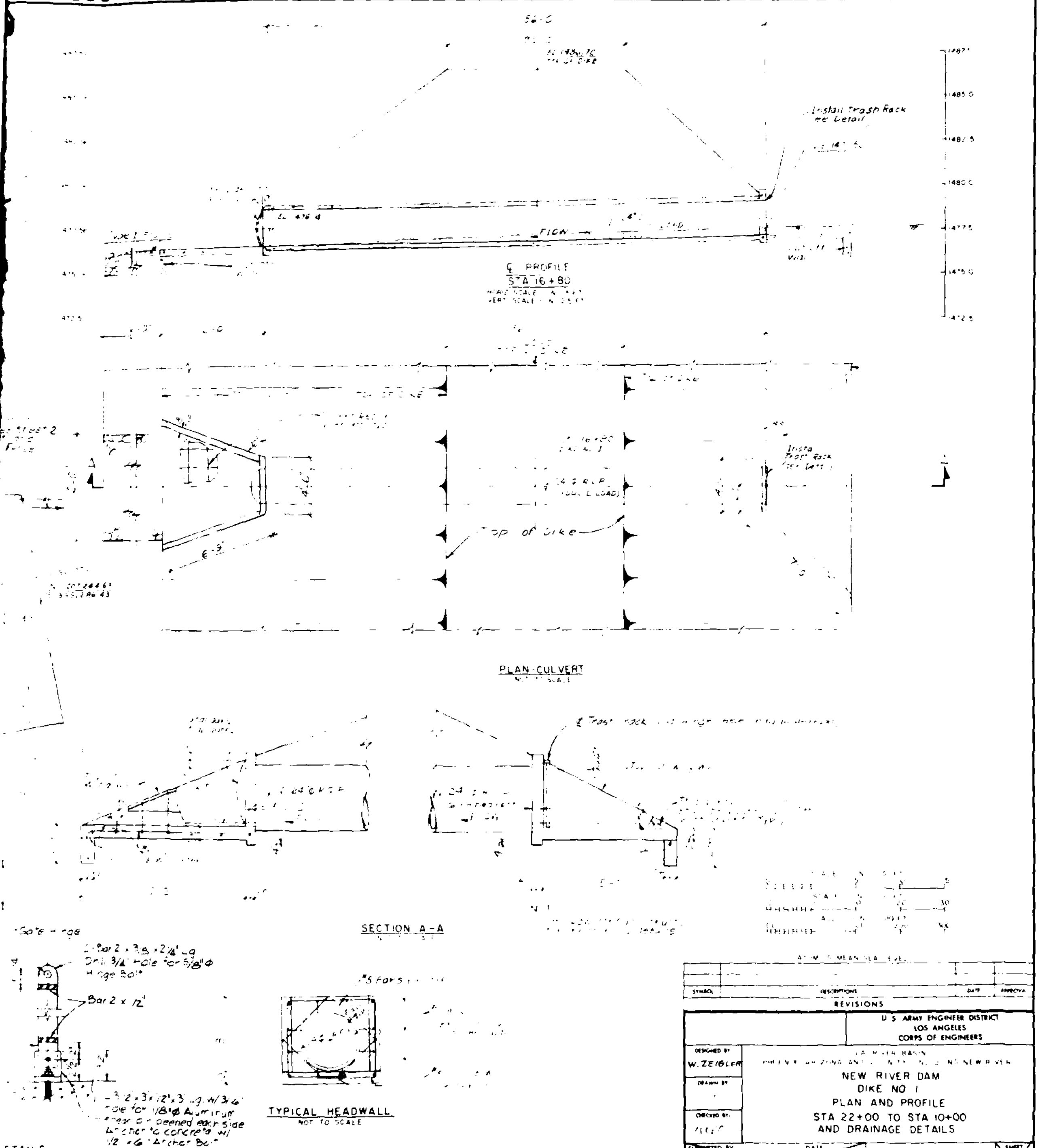
VALUE ENGINEERING PAYS



STRUCTURAL DETAILS

SAFETY PAYS

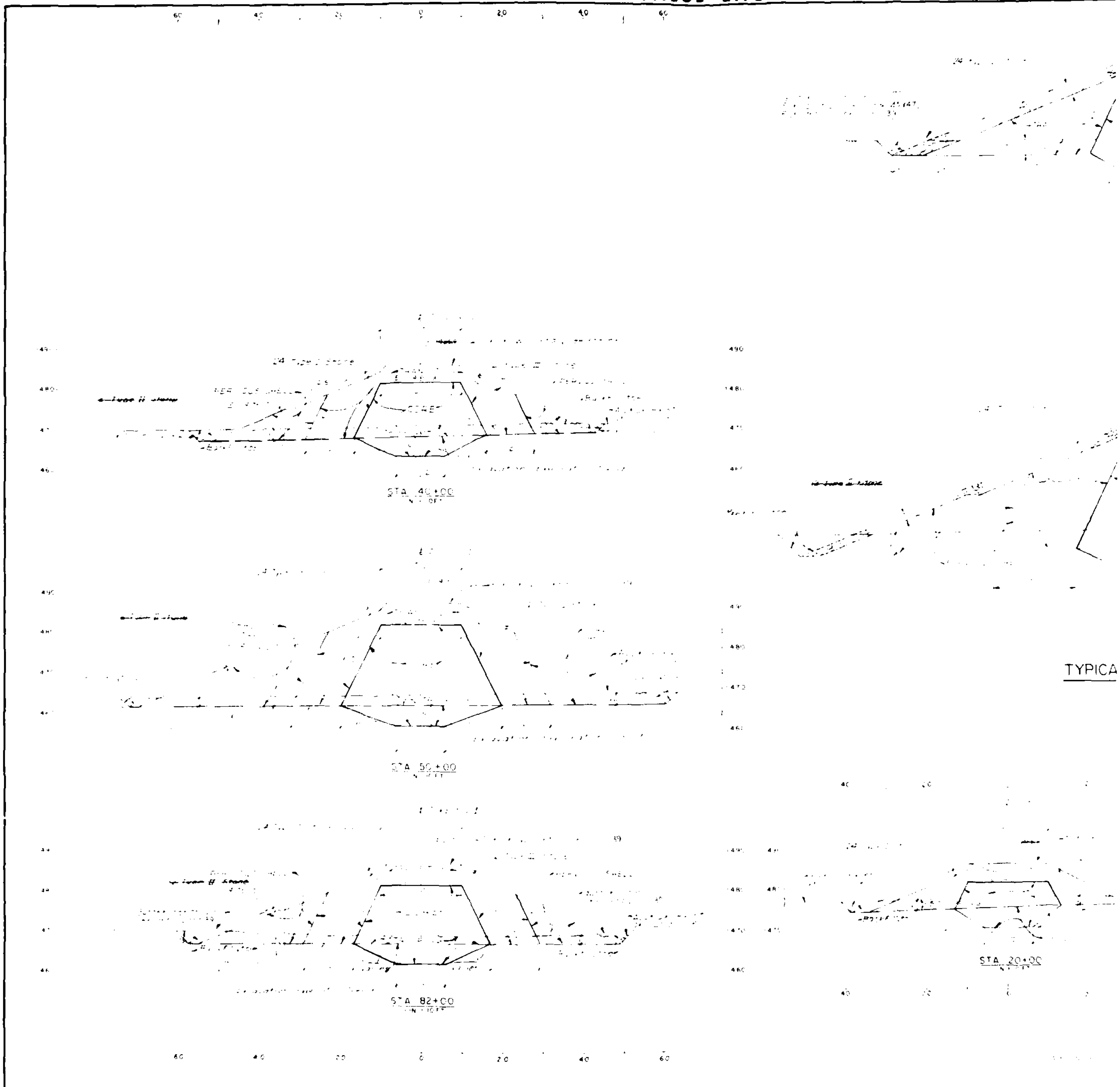
VALUE ENGINEERING PAYS

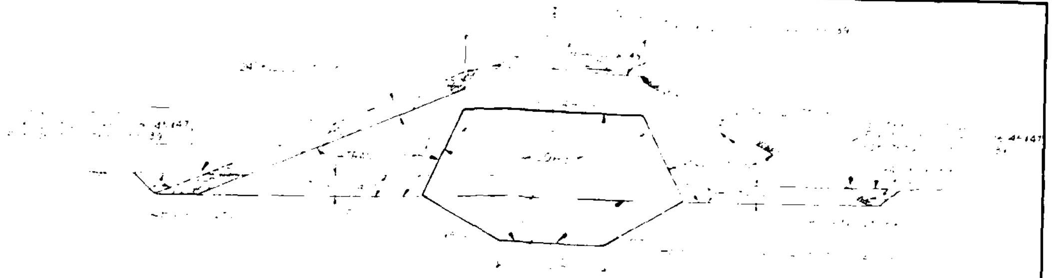


DESIGNED BY	W. ZEIGLER
	U.S. ARMY ENGINEER DISTRICT LOS ANGELES CORPS OF ENGINEERS
	NEW RIVER DAM DIKE NO 1 PLAN AND PROFILE STA 22+00 TO STA 10+00 AND DRAINAGE DETAILS
SPEC NO	DACW09-83-B-0016
DISTRICT FILE NO	24/272
SHEET	80

REVISIONS

SAFETY PAYS

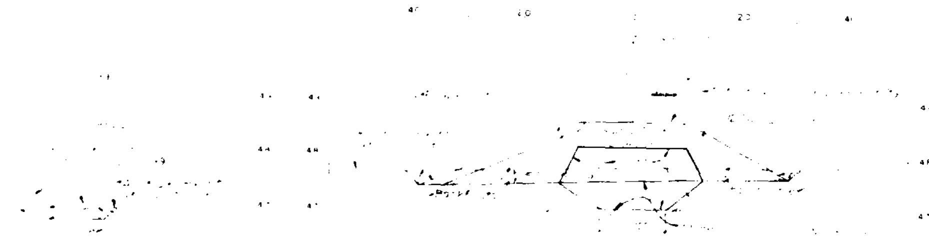

STA 40+00
STA 50+00
STA 82+00
STA 20+00
TYPICA



STA 84+75± TO STA 82+75
STA 35+00 TO STA 10+00±

STA 82+75 TO STA 35+00

TYPICAL SECTIONS - DIKE NO. I



STA 20+00

REVISIONS

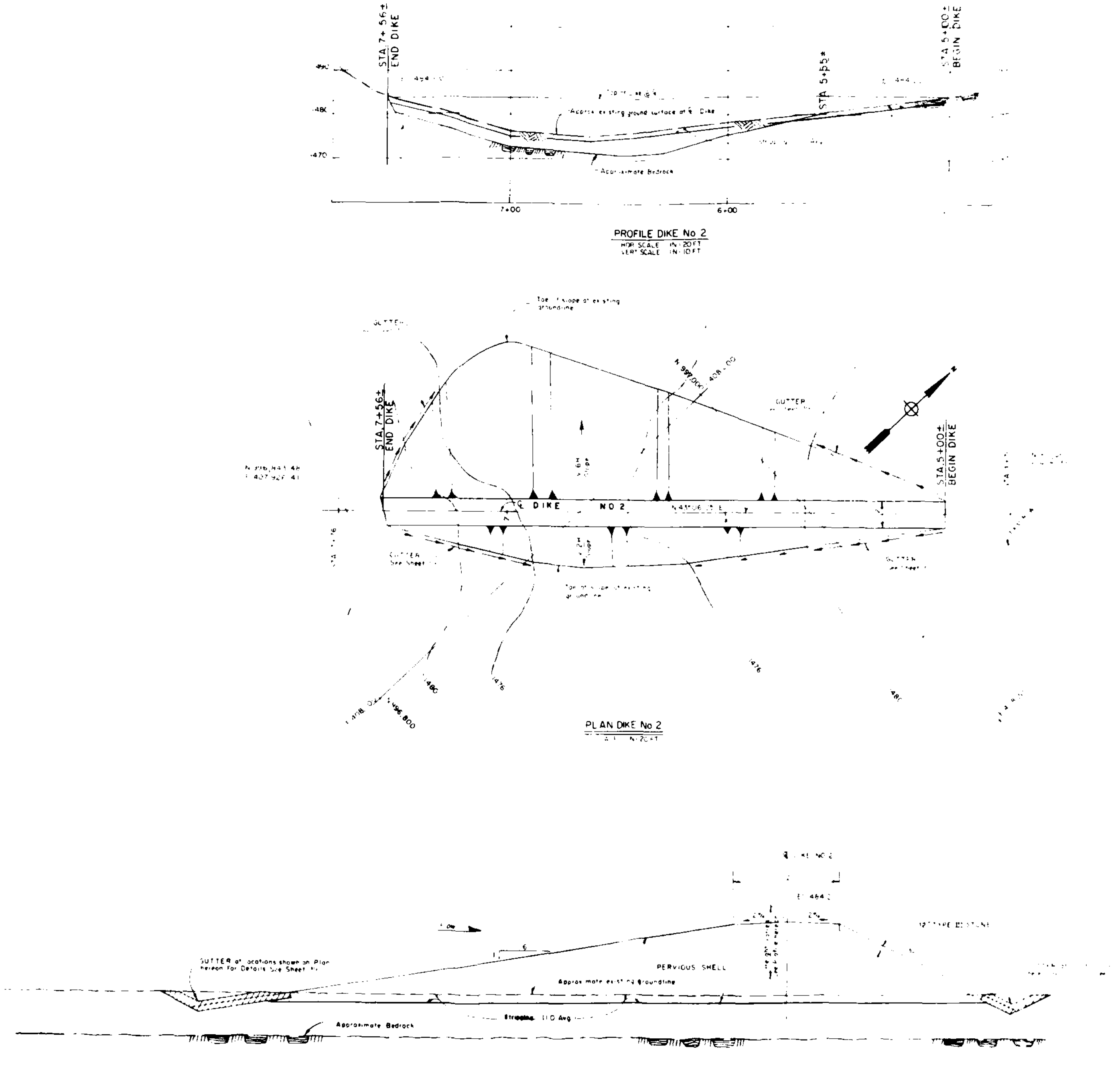
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

NEW RIVER DAM

DIKE No. 1

CROSS SECTIONS AND TYPICAL SECTIONS

DESIGNED BY | DRAWN BY | CHECKED BY | SUBMITTED BY | DATE APPROVED | SPEC. NO. DACW09 | SHEET

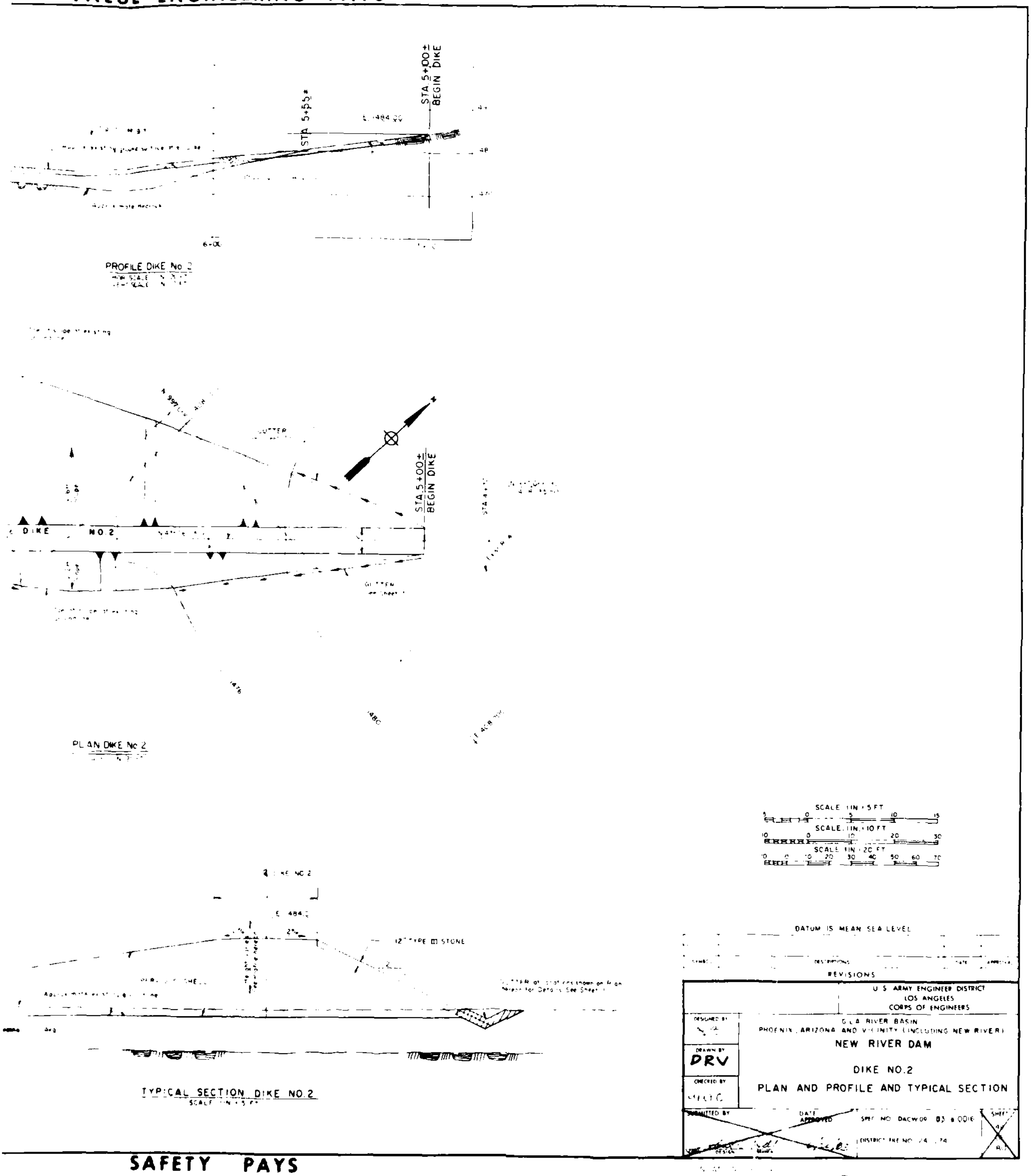


PROFILE DIKE No 2

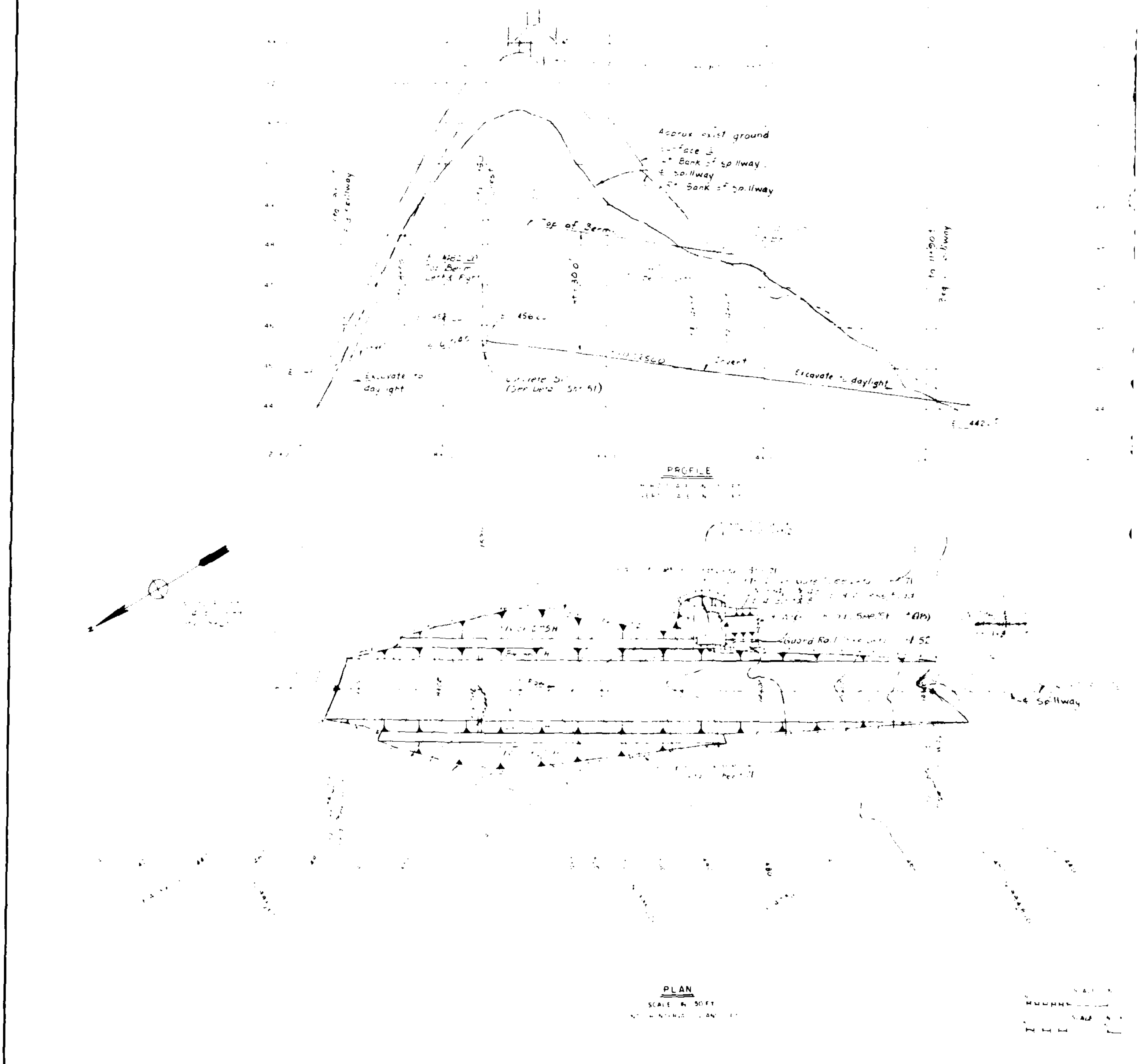
PLAN DIKE No 2

TYPICAL SECTION DIKE NO 2

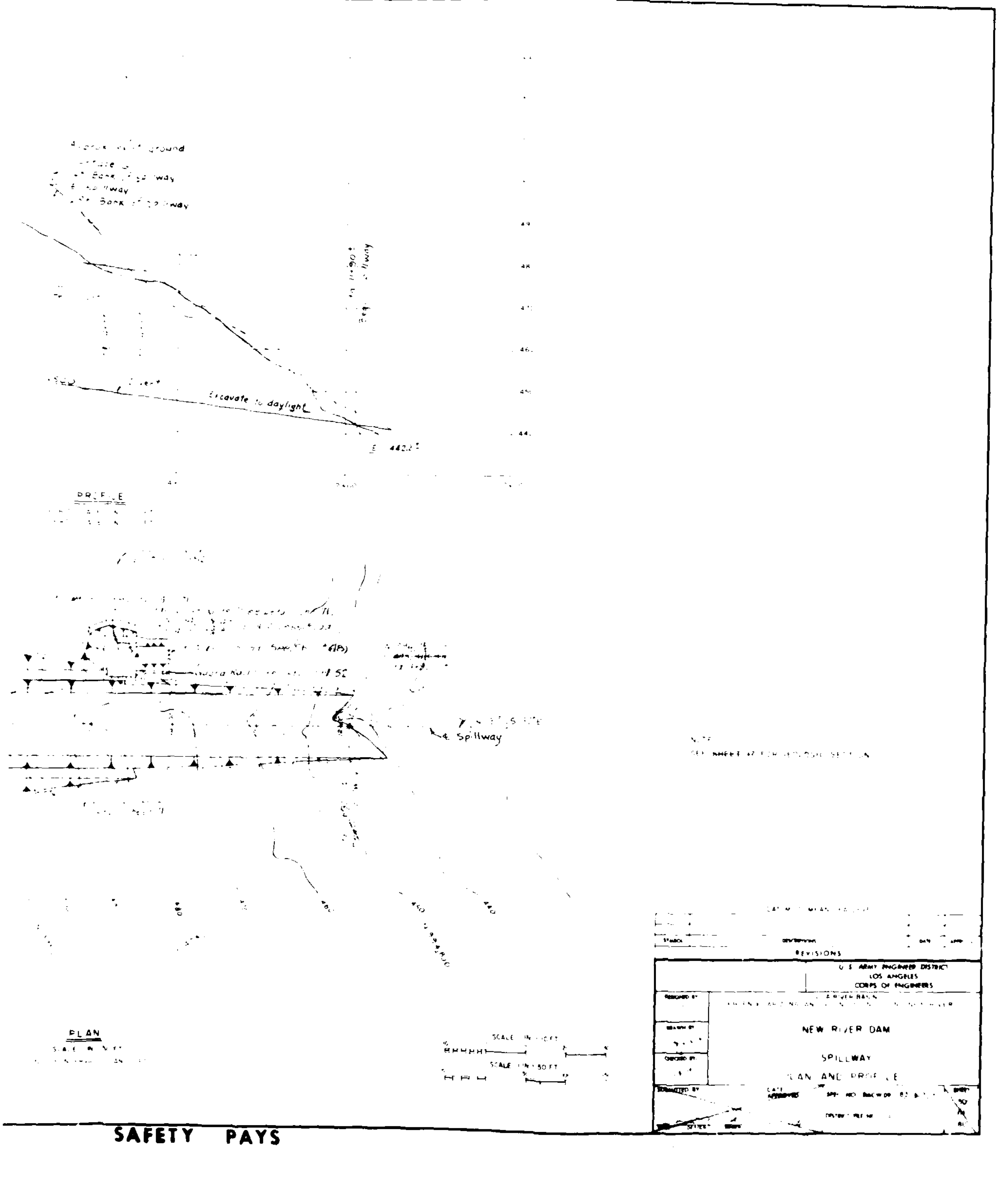
VALUE ENGINEERING PAYS



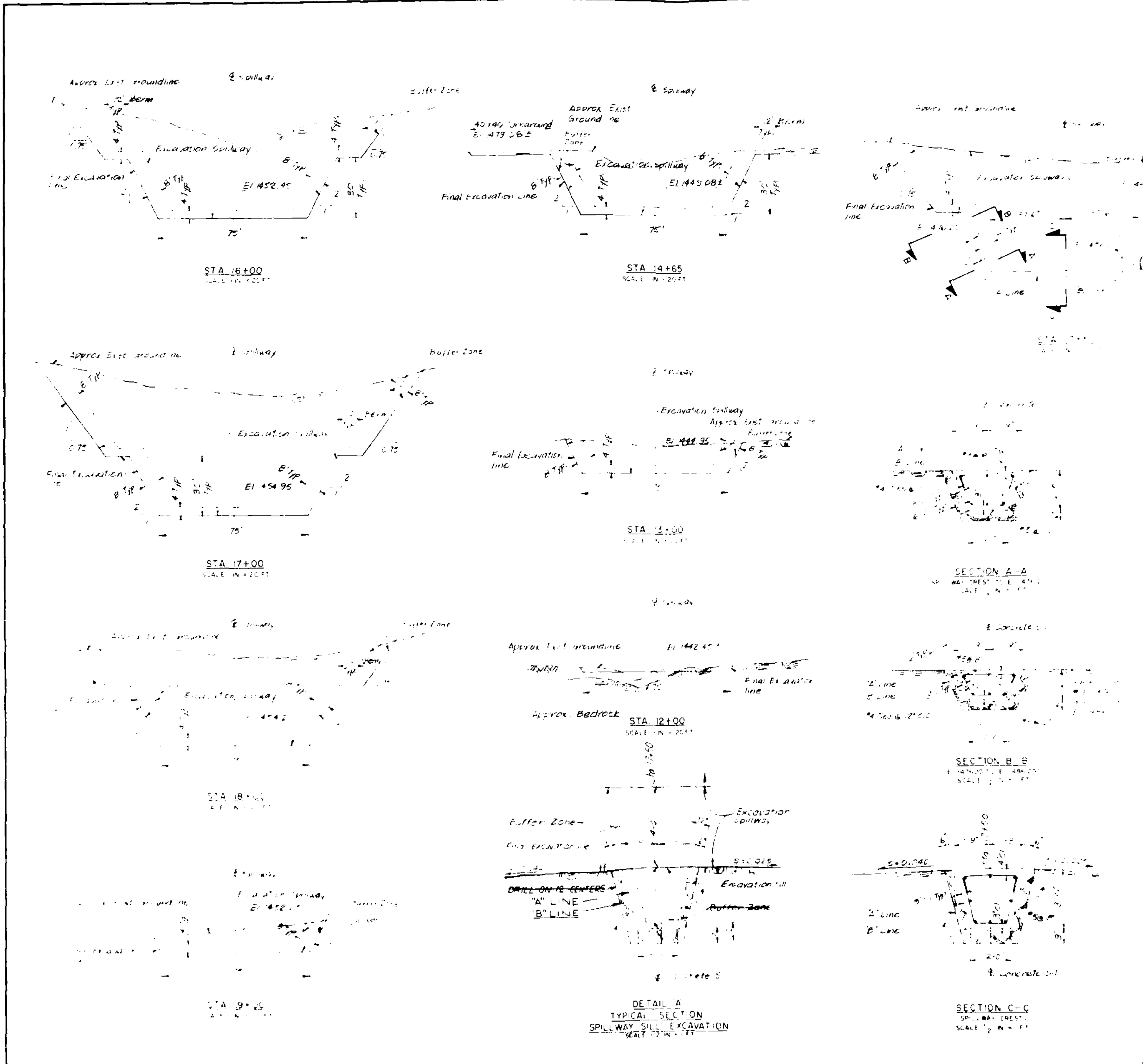
SAFETY PAYS


Approx exist ground
surface
Bank of spillway
Spillway
Top of Berm
Invert
Excavate to daylight
Excavate to daylight
PROFILE
Guard Rail
Spillway
PLAN
SCALE 1 IN 50 FT

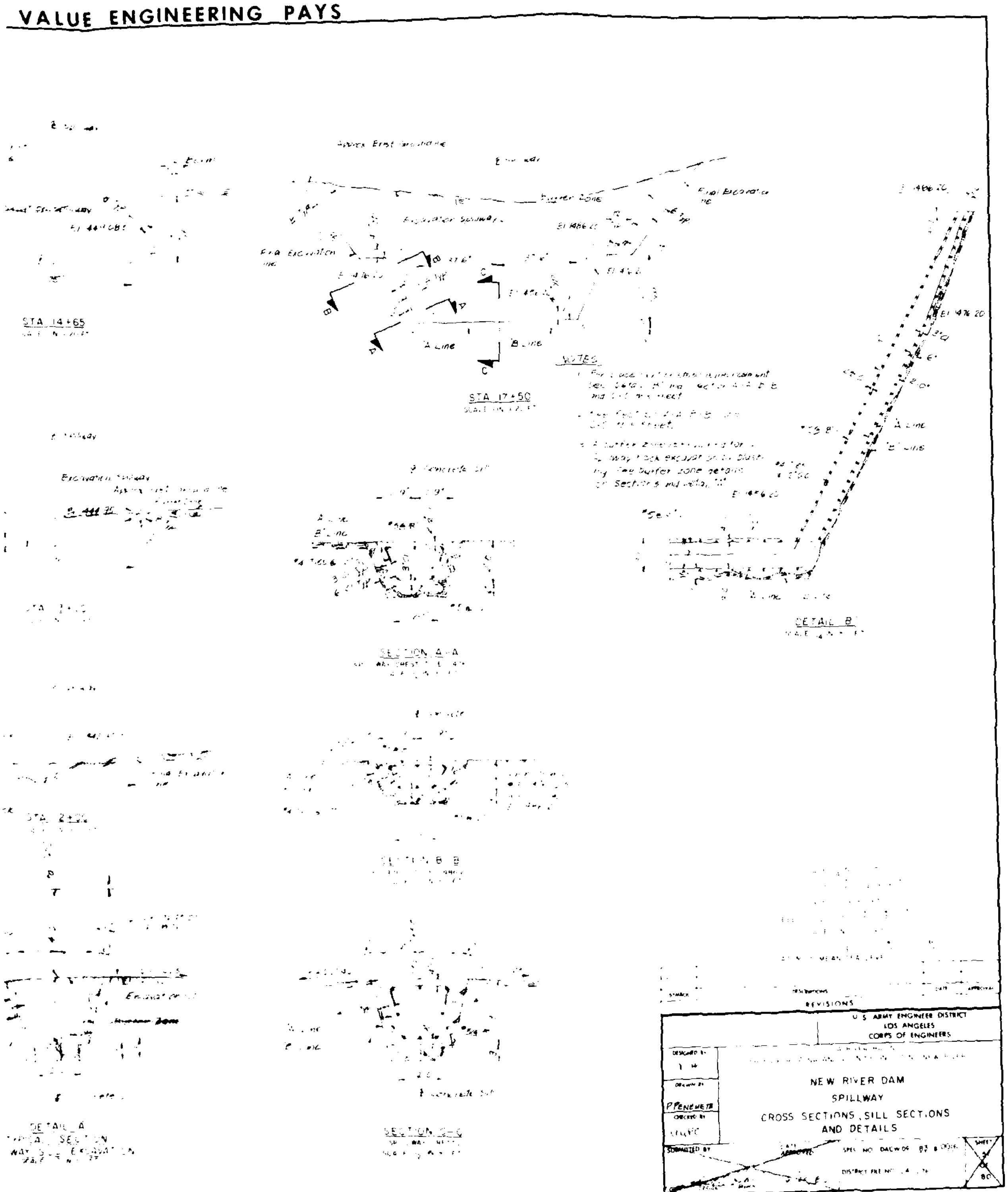
VALUE ENGINEERING PAYS



SAFETY PAYS

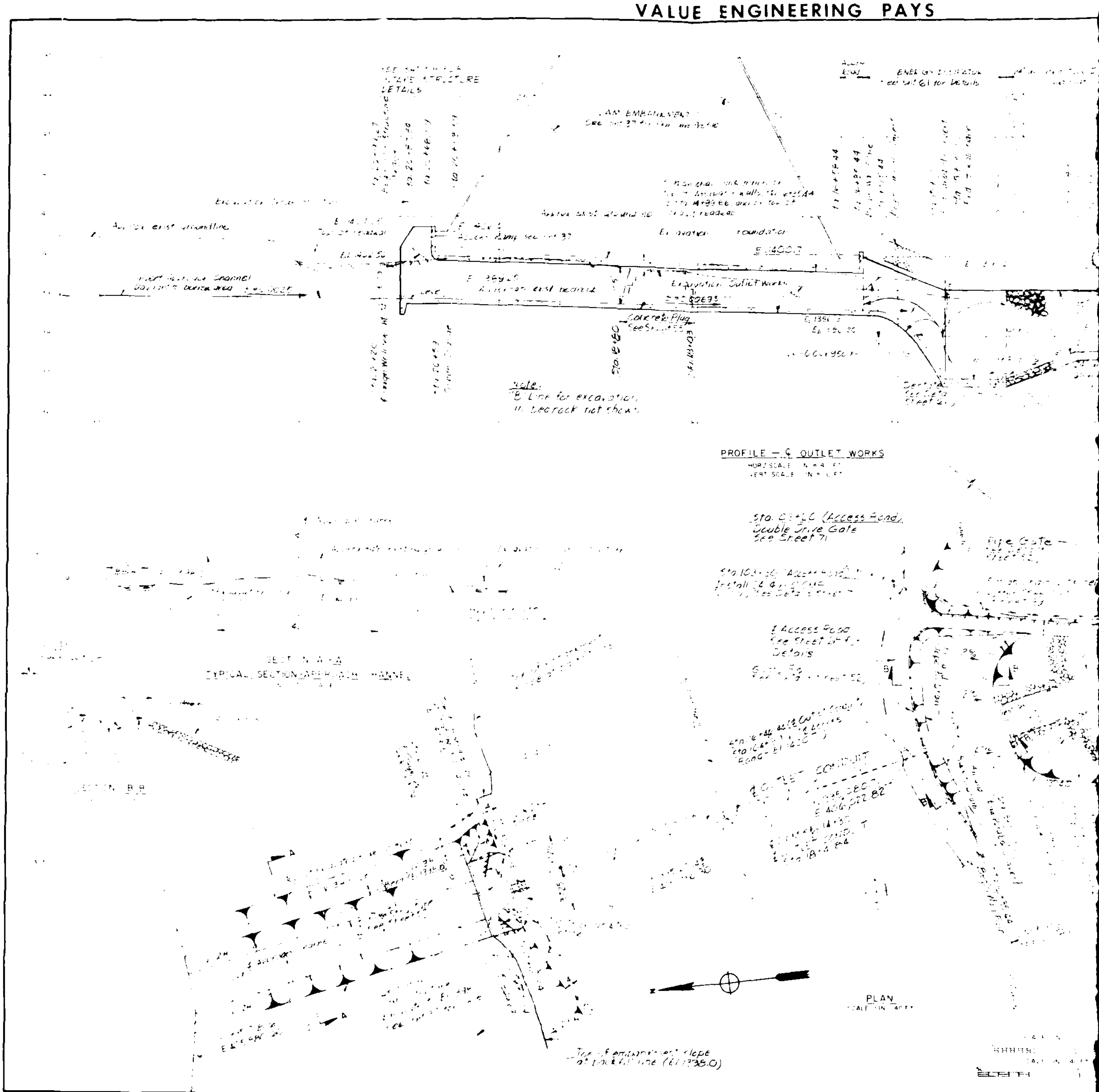

STA 16+00
El 1452.45
75'
Excavation Spillway
STA 14+65
El 1449.08±
Buffer Zone
Final Excavation Line
STA 17+00
STA 12+00
Approx. Bedrock
SECTION A-A
SECTION B-B
SECTION C-C
"A" LINE
"B" LINE
DETAIL A
TYPICAL SECTION
SPILLWAY SILL EXCAVATION

VALUE ENGINEERING PAYS



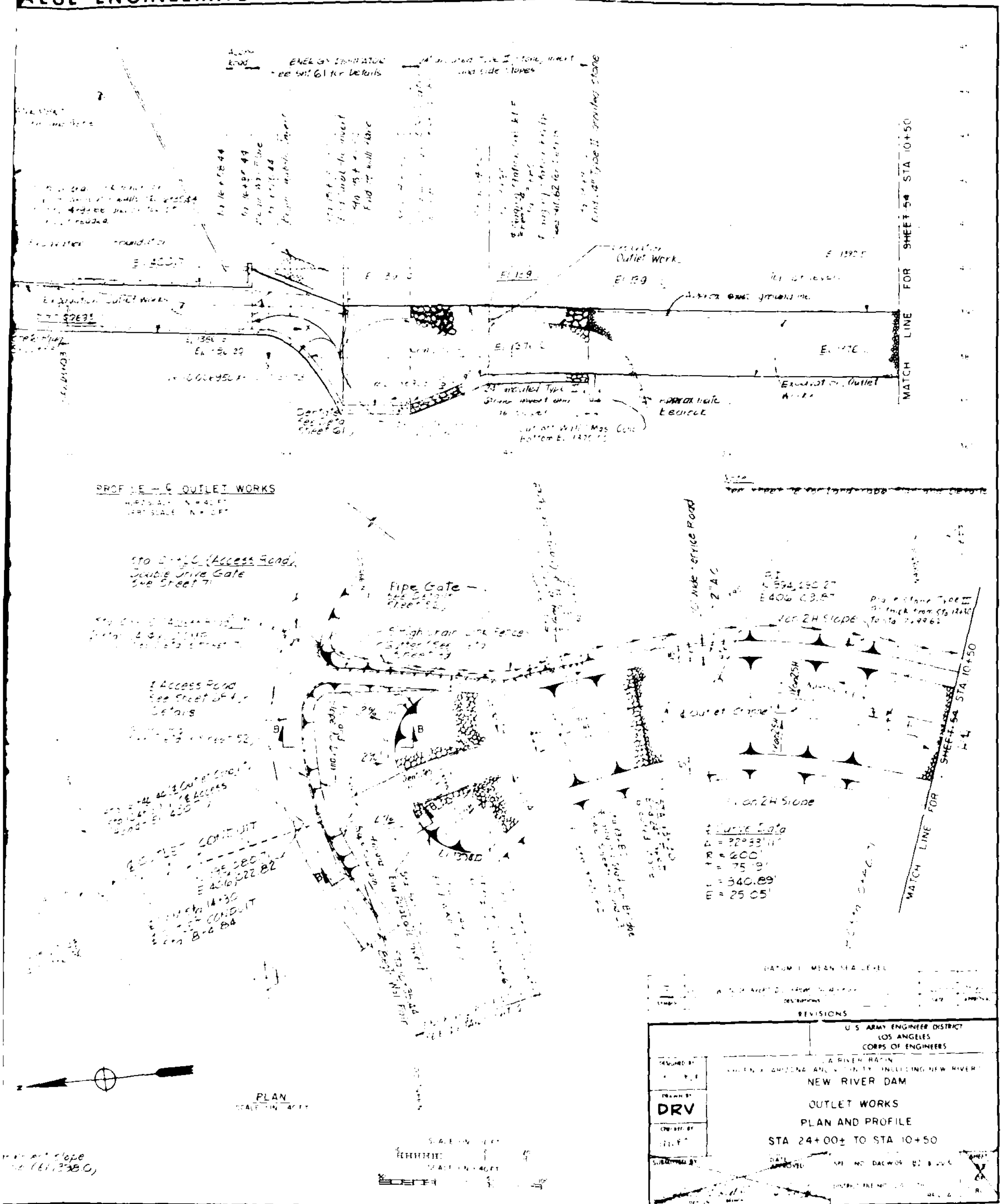
SAFETY PAYS

VALUE ENGINEERING PAYS



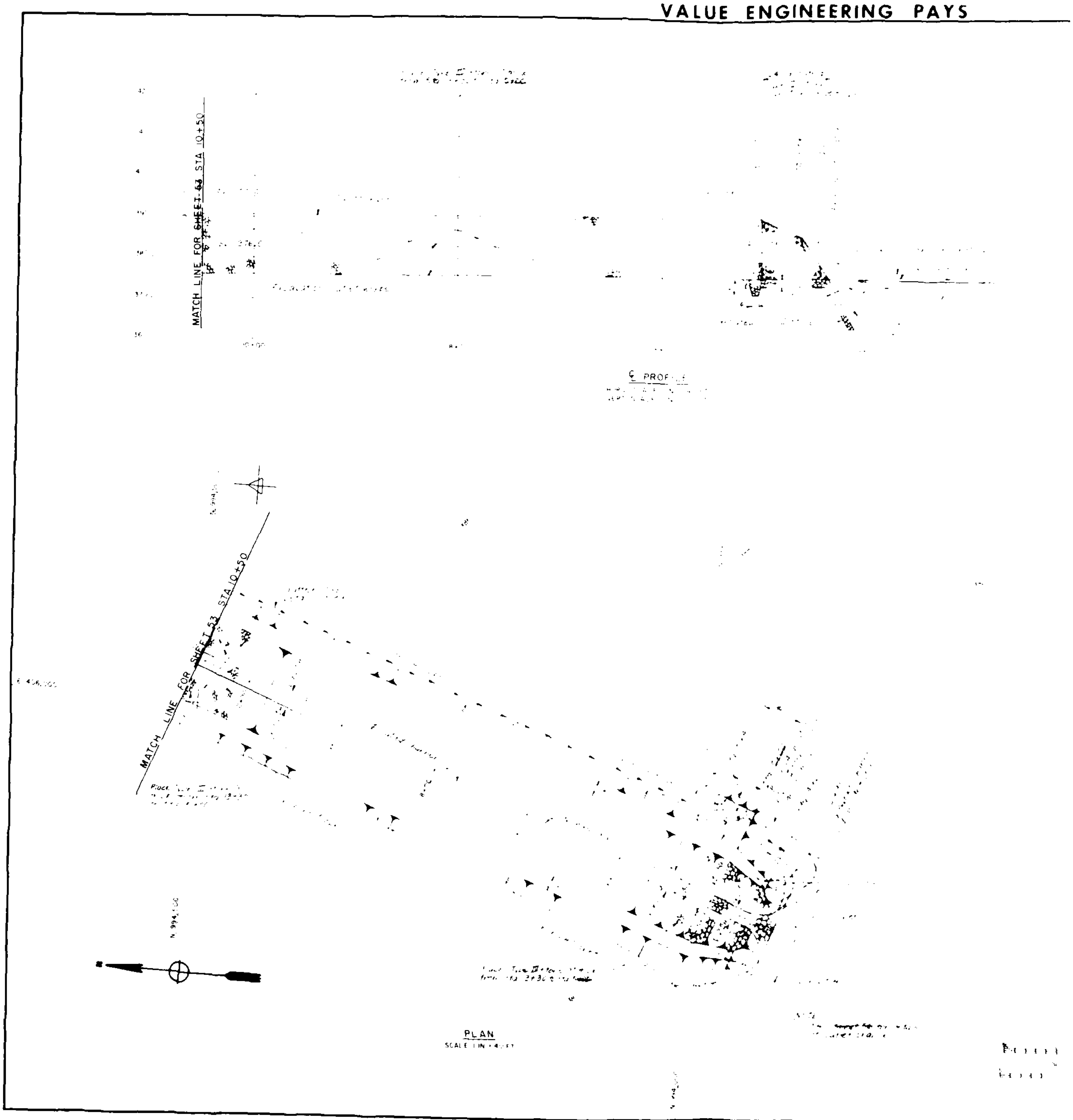
SAFETY PAYS

VALUE ENGINEERING PAYS



SAFETY PAYS

VALUE ENGINEERING PAYS



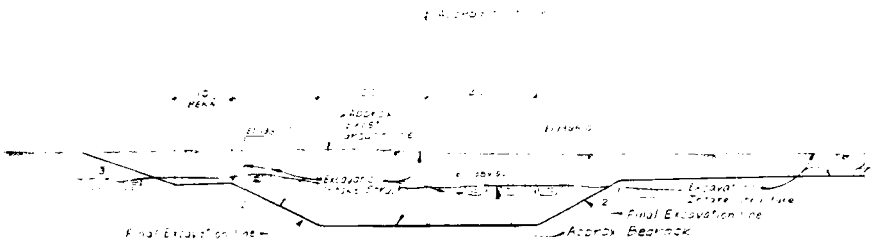
SAFETY PAYS

VALUE ENGINEERING PAYS

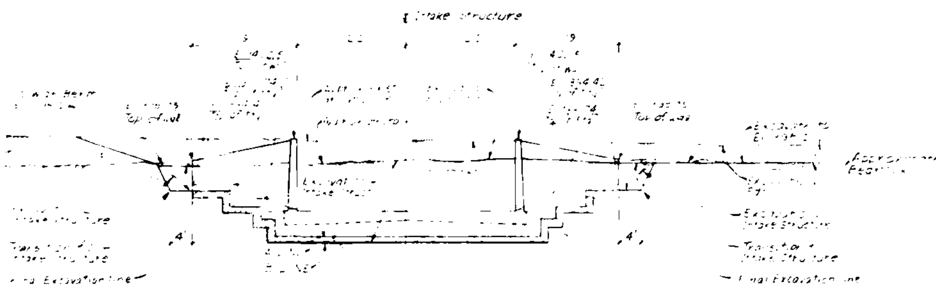
	U S ARMY ENGINEER DISTRICT LOS ANGELES CORPS OF ENGINEERS
DESIGNED BY	NEW RIV R DAM
DRAWN BY	OUTLET WORKS
CHECKED BY	PLAN AND PROFILE STA 10+50 TO STA 4+35
SUBMITTED BY	DATE APPROVED
SPEC NO DACW09 83 B 0016	SHEET OF 80
DISTRICT FILE NO 24 [illegible]	

REVISIONS

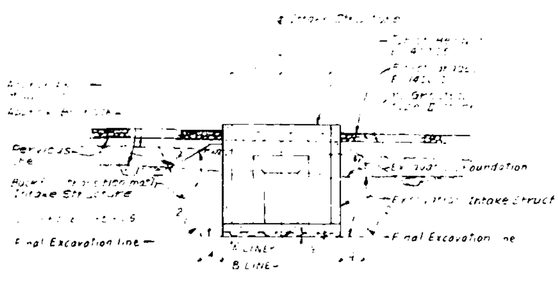
SAFETY PAYS



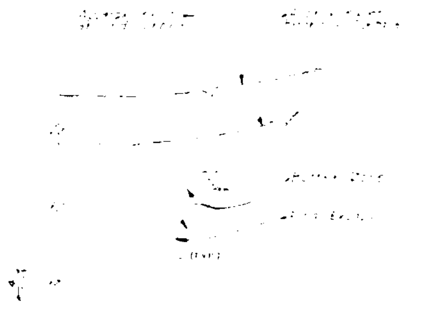
STA. 22+00



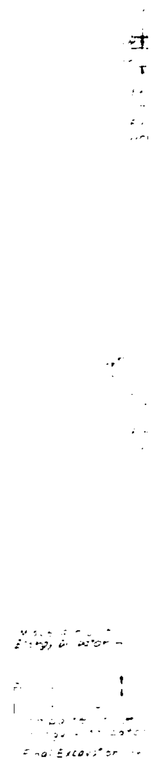
STA. 20+95.27



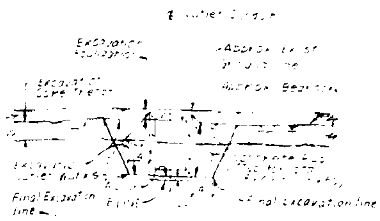
STA. 20+68.59



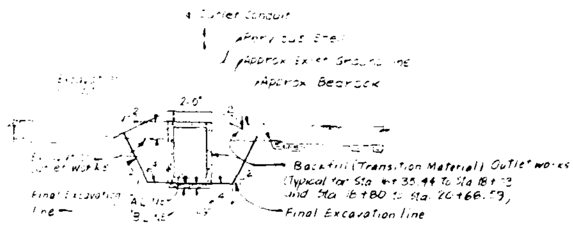
BUFFER ZONE DETAIL
TYPICAL SECTION
NOT TO SCALE



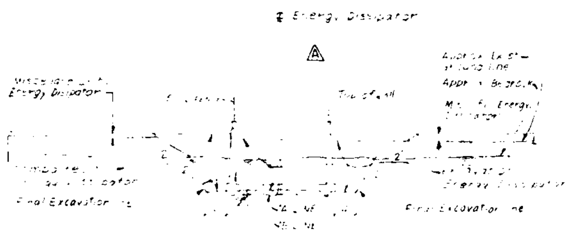
VALUE ENGINEERING PAYS



STA. 18+41.84

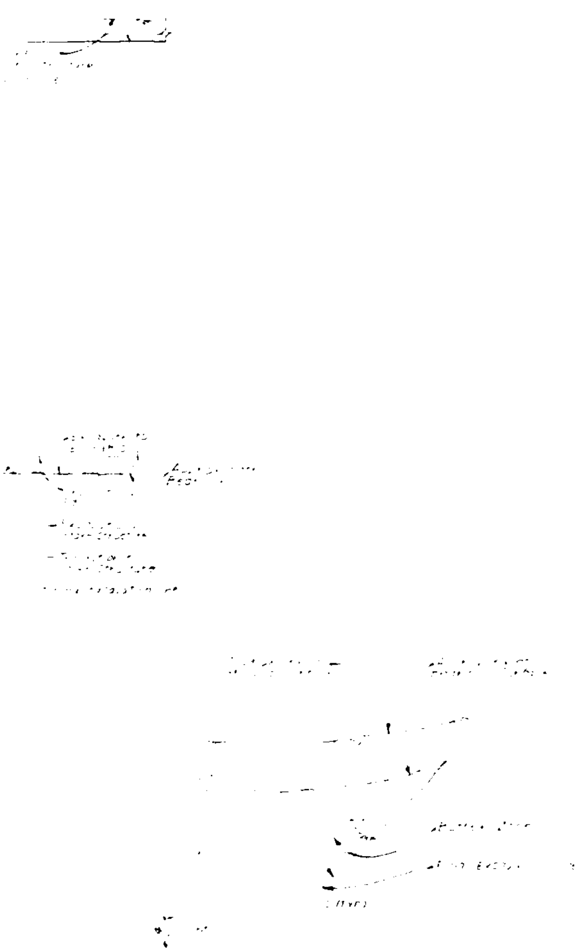


STA. 17+00



STA. 16+00

NOTE



BUFFER ZONE DETAIL
TYPICAL SECTION
NOT TO SCALE

DATUM IS MEAN SEA LEVEL

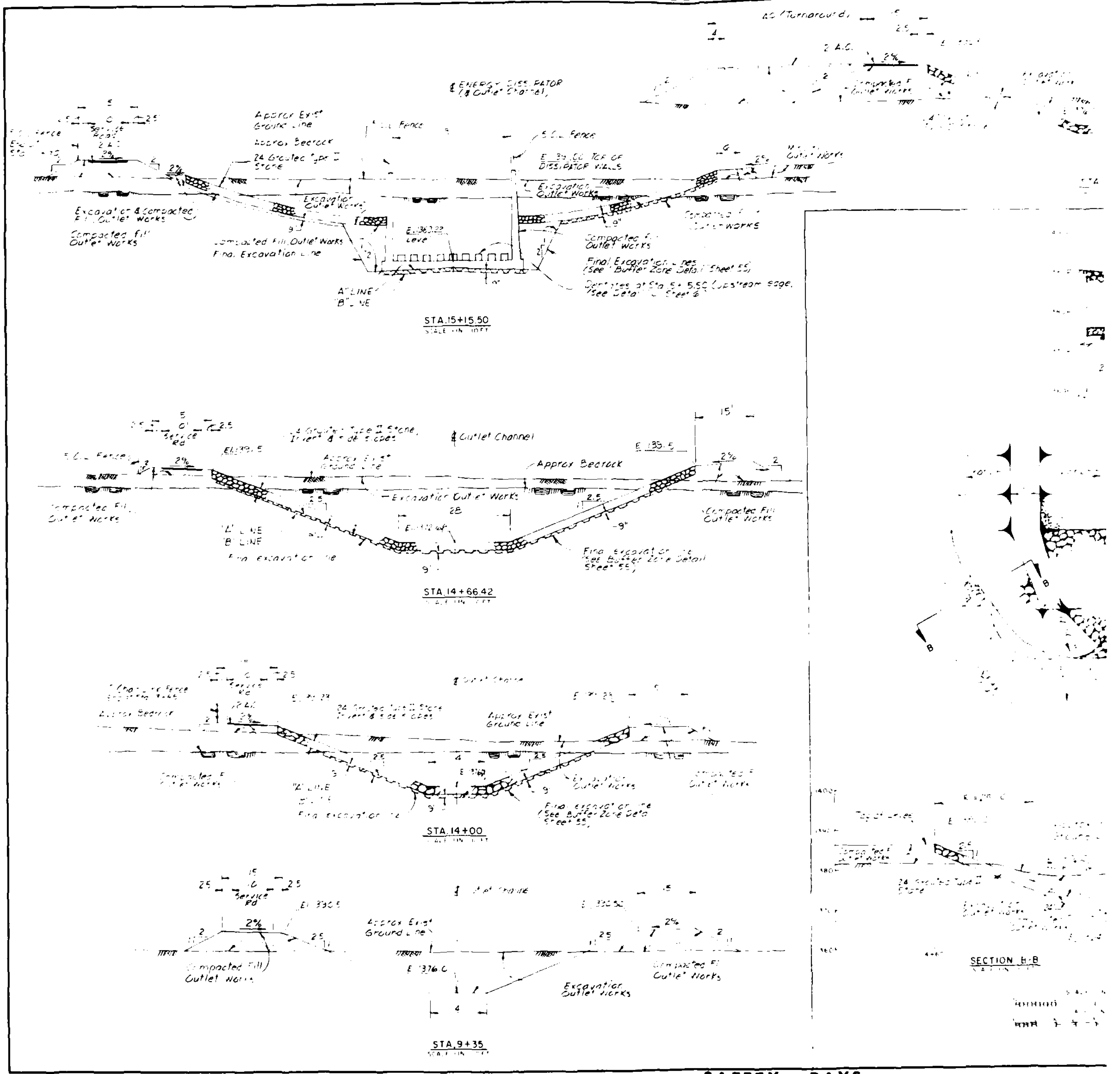
REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

GILA RIVER BASIN
PHOENIX, ARIZONA AND VICINITY (INCLUDING NEW RIVER)
NEW RIVER DAM
OUTLET WORKS
CROSS SECTIONS

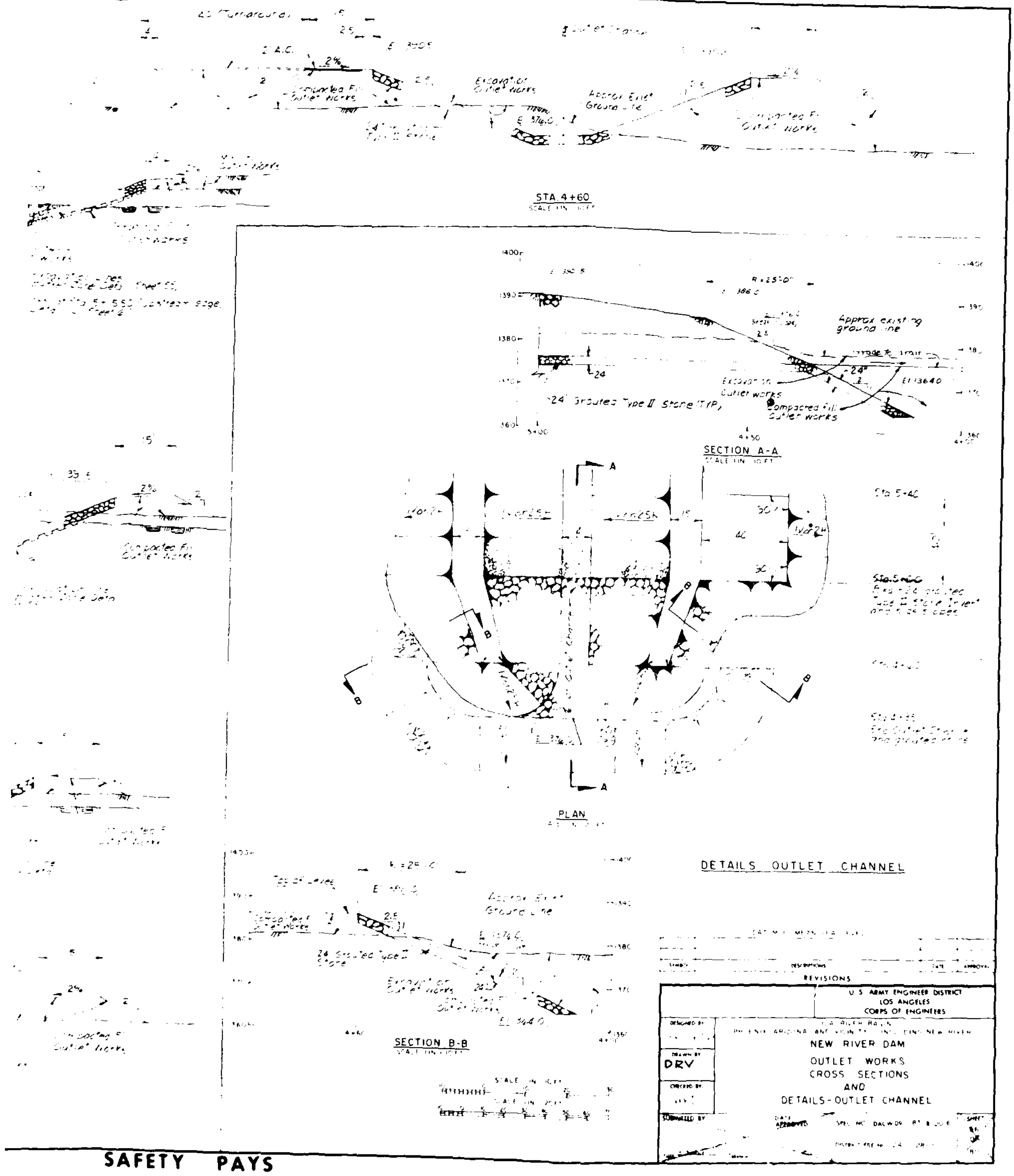
SAFETY PAYS

VALUE ENGINEERING PAYS

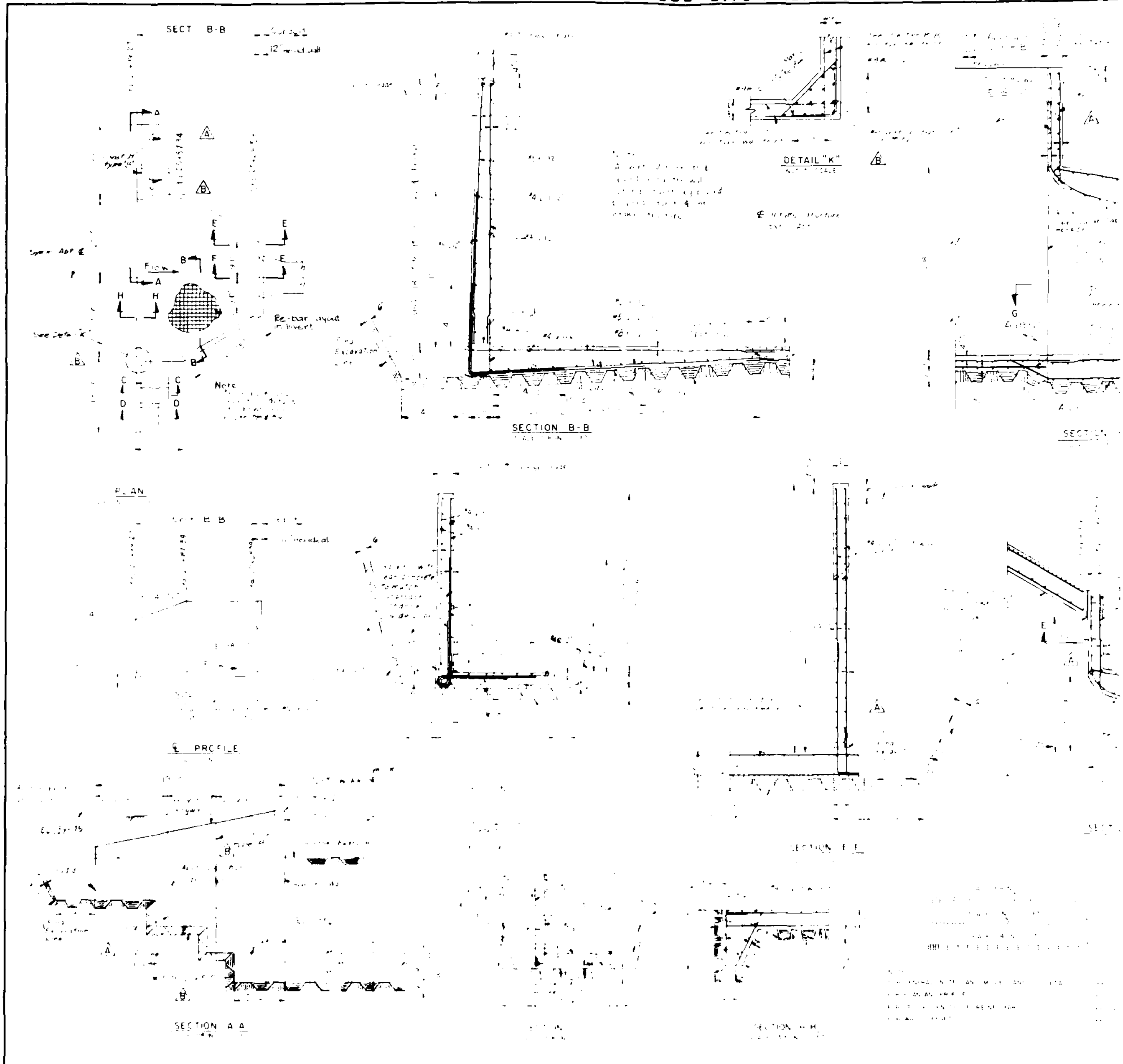


SAFETY PAYS

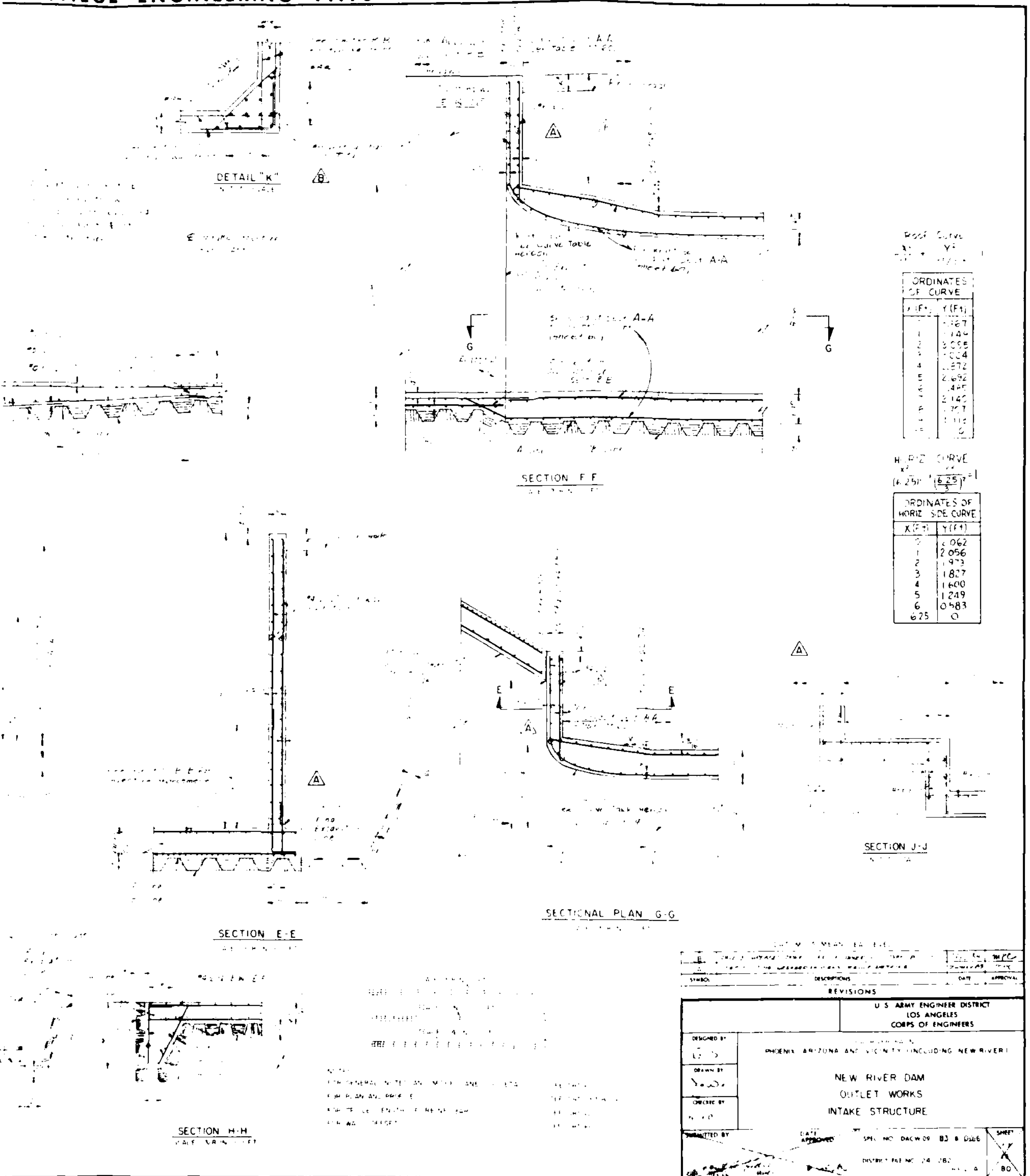
VALUE ENGINEERING PAYS



SAFETY PAYS


SECT B-B
DETAIL "K"
PLAN
Flow
Re-bar ...
SECTION B-B
℄ PROFILE
SECTION E-E
SECTION A-A

VALUE ENGINEERING PAYS



ORDINATES OF CURVE

X (Ft)	Y (Ft)
[illegible]	[illegible]67
1	[illegible]149
2	3 095
[illegible]	[illegible]004
4	[illegible]872
5	2.692
[illegible]	[illegible]485
[illegible]	2 140
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	0

ORDINATES OF HORIZ SIDE CURVE

X (Ft)	Y (Ft)
0	[illegible] 062
1	2 056
2	1 973
3	1 827
4	1 600
5	1 249
6	0 583
6 25	0

SAFETY PAYS

ATTACHMENT 1

DEPARTMENT OF THE ARMY
SOUTH PACIFIC DIVISION, CORPS OF ENGINEERS
LABORATORY

PETROGRAPHIC ANALYSIS OF FOUNDATION ROCKS

NEW RIVER DAM

SAUSALITO, CALIFORNIA

June 1984

GENERAL TEST REPORT	DATE ? JUN 1984 DISTRICT LOS ANGELES	SPD LABORATORY CORPS OF ENGINEERS, US ARMY SAUSALITO CALIFORNIA 94966
PROJECT NEW RIVER DAM PHOENIX, ARIZONA	CONTRACT No.	WORK ORDER No. & DATE 20 APRIL 1984 CIV 84-87
UNIT COST $?.300.00	DATE SAMPLE RECEIVED 6 APRIL 1984	LABORATORY No. NRD 1 TO 6
DESCRIPTION PETROGRAPHIC ANALYSIS OF FOUNDATION ROCKS	SOURCE	NEW RIVER DAM
PETROGRAPHER PETER ALMENDINGER		

FINDINGS

NDR-1 Greisen (pneumatolytic altered alkal-feldspar or granite)

MACROSCOPIC

The sample is dark gray (fresh) to medium gray (weathered) with calcite coating up to 2mm thick, relatively fresh and well indurated. The principle constituents include fine grained interlocking crystals of quartz, plagioclase and alkali feldspar, and biotite. The rock is massive, hard and shows no microscopic fracturing. The quartz, plagioclase and biotite give the rock a mottled appearance.

MICROSCOPIC

Composed essentially of fine to medium grained crystals at approximately 50-60% equigranular quartz, 30-40% feldspars (plagioclase and microciine), 10-15% olive green biotite, 5-10% colorless muscovite, 1-2% aegirine-augite, and trace of magnetite. The sample represents granitic material which has undergone postmagmatic alteration by residual aqueous-gaseous fluids that continue to rise through the intergranular pore spaces. The quartz, plagioclase, microcline, biotite and aegirine-agate show solution etching forming irregular masses scattered throughout the sample. Quartz has been severely etched and has been redeposited as smaller anhedral equigranular grains bounded by plagioclase, alkali feldspars, and biotite. The subhedral to euhedral plagioclase and alkali feldspars have been severely altered with muscovite forming at the expense of the feldspars. The smaller grains of plagioclase have been completely altered to muscovite. Colorless aegirine-augite and greenish biotite are found intergrown and form isolated masses throughout the sample.

NDR-2 Diorite

MACROSCOPIC

The sample is medium gray (fresh) to whitish-gray and medium gran (weathered), relatively fresh and well indurated. The principle constituents include medium to coarse grained interlocking crystals of quartz, plagioclase feldspar, hornblende, and biotite. The rock is massive, hard, and shows no fracturing. The quartz, plagioclase, and biotite give the rock a mottled appearance.

MICROSCOPIC

Composed essentially of medium to coarse grained,anhedral to euhedual intergranular crystals of approximately 60-70% plagioclase feldspar of andesine composition, 5-10% anhedral, non-undulated quartz, 20-25% subhedral biotite, 10-15% subhedral to euhedral hornblende, and minor clinopyroxene, zircon, chlorite, sphene. The sample as a whole is relatively fresh with no alternation of the feldspars or mafic minerals. The plagioclase feldspars are characterized by interlocking medium to coarse grained elongated subhedral to euhedral laths, and An content of approximately 15-22%. The plagioclase laths are interlocked with anhedral grains of quartz which form intergranular masses. The mafic and accessory mineral are associated together and form lenticular masses throughout the sample. Yellowish-brown to brownish biotite and greenish hornblende are well preserved as subhedral to euhedral crystals. Zircon, sphene, and clinopyroxene from minor constituents of the hornblende-biotite masses.

NRD-3 Quartz-rich granitoid (dike rock)

MACROSCOPIC

The sample is medium gray to black (fresh) to light gray (weathered), aphanitic, massive, and generally hard. There is no preferred fracturing orientation and contains scattered large crystals in the aphanitic host rock. The larger crystals are light gray and are subhedral in shape. The principle constituents include very fine grained crystals of quartz, hornblende, and biotite.

MICROSCOPIC

Sample consists of anhedral to subhedral, equigranular *(of the same size)* crystals of approximately 40-50% anhedral quartz, 30-40% greenish, anhedral to subhedral hornblende, 20-30% brownish subhedral biotite, and 0-10% anhedral plagioclase feldspar. The grain size ranges from .1 to .5mm (very fine), with the hornblende and biotite grains being slightly larger than quartz and plagioclase. There is a slight preferred orientation of the hornblende and biotite crystal in one direction. There are sparsely scattered phenocryst-like crystals of quartz, biotite, hornblende, and plagioclase which have a rim of smaller hornblende and biotite crysatls. The texture and association with other lithologies suggests that it is an igneous dike. Petrographic evidence includes: the high amount of quartz, slight preferred orientation of mineral grains, the small grain size suggesting rapid cooling as compared to the surrounding igneous rock, and the igneous-like texture.

NRD-4 Granite

MACROSCOPIC

The sample is light gray (fresh and weathered), moderately weathered, and moderately hard. The principle constituents include medium to coarse grained interlocking crystals of quartz, plagioclase, microcline, and minor amount of biotite. Calcite commonly coats the weathered surface of the sample. The granite contains numerous fractures which penetrate throughout the sample causing the rock to be broken apart easily.

MICROSCOPIC

Sample consists of coarse grained, anhedral to euhedral crystals of approximately 40-50% undulated, anhedral quartz, 40-50% alkali feldspar, chiefly microcline, 15-20% plagioclase feldspar of albite composition, 0-5% yellowish-brown to brown biotite and colorless muscovite, and accessory minerals consisting of magnetite, zircon, and allanite (epidote group).

The sample as a whole is moderately weathered indicated by the feldspar grains but weathering has not affected the mafic minerals. Quartz is characterized by anhedral interlocking grains filling the institial spaces between feldspar grains. The dominant alkali feldspar is subhedral to euhedral microcline with grains reaching 8mm, and exhibits partial replacement by muscovite and sericite. Albite is the dominant plagioclase feldspar and as in the microcline, muscovite is present especially along grain boundaries. The biotite, muscovite, and accessory mineral occurs in isolated masses randomly scattered throughout the sample.

NRD-5 Porphyritic Andesite

MACROSCOPIC

The sample is pinkish-gray (fresh and weathered), exhibits little weathering and is moderately hard. The principle constituents include fine grained euhedral, phenocrysts of hornblende, biotite, and plagioclase in a aphanitic perlitic glassy matrix. On the weathered surface, texture exhibits concentric weathering the glassy matrix, as the glass along the concentric fractures have weathered more rapidly leaving a hilly topgraphy. The phenocrysts are randomly oriented and are dispersed randomly throughout the sample.

MICROSCOPIC

Sample consists of subhedral to euhedral, fine grained (.2-1.0mm) phenocrysts of plagioclase, hornblende, biotite, and magnetite in a perlitic glassy matrix. The perlitic texture is breakage along concentric shells which is in response to hydration of the glass and its consequent expansion. The phenocryst composition is 65-75% plagioclase feldspar mainly of oligoclase composition (An 5-10), 0-5% alkali feldspar (orthoclase), 15-20% subhedral to euhedral brownish oxyhornblende (basaltic hornblende), 15-20% brownish euhedral biotite, and 0-5% euhedral magnetite. The plagioclase feldspar phenocrysts occurs as zoned and unzoned twinned crystals. This would suggest that the rock had undergone two periods of crystallization. The unzone plagioclase would represent crystallization at depth, while the zoned plagioclase would represent crystallization during upward movement or at eruption. Hornblende is yellow-brown to reddish-brown, forms euhedral crystals and is associated with subhedral brownish biotite. Magnetite occurs as subhedral to euhedral (.1mm) sparsely scattered throughout the sample.

NRD-6 Porphyritic Andesite

MACROSCOPIC

The sample is reddish-brown (fresh and weathered) exhibits little weathering and is moderately hard. The priciple constituents include large (medium grained) whitish phenocrysts of plagioclase, fine grained euhedral phenocrysts of hornblende, biotite, and magnetite in an altered glassy matrix. The phenocrysts are randomly oriented and are dispersed randomly through the sample. There are several fractures filled with whitish chalcedony/agate material ranging up to several mm thick.

MICROSCOPIC

Sample consists of subhedral to euhedral, fine to medium grained phenocrysts of plagioclase, common and oxyhornblende, biotite, and magnetite in an altered glassy matrix. The phenocrysts composition is approximately 60-65% subhedral to euhedral plagioclase feldspar mainly of

andesine composition (An 15-20), 2-3% common euhedral green hornblende, 5-6% subhedral to euhedral, yellowish-brown to reddish brown oxyhornblende (basaltic hornblende), 20-22% brownish subhedral biotite, and 4-6% euhedral black magnetite. The plagioclase feldspars phenocrysts occur as both zoned crystals and as single unzoned elongated laths up to 3mm as in NRD-5. Biotite is subhedral, brownish in color and as in the hornblende phenocrysts never exceed 1mm in size. Magnetite occurs as subhedral to euhedural phenocryst up to .4mm sparsely scattered throughout the sample.

NOTE: Samples NRD-1 and NRD-2 are most likely of the same origin. The differences are that NRD-1 represents material near the margins of the Diorite pluton which is represented by NRD-2. NRD-1 is the postmagmatic alternation of NRD-2.

ATTACHMENT 2

DEPARTMENT OF THE ARMY
SOUTH PACIFIC DIVISION, CORPS OF ENGINEERS
LABORATORY

PETROGRAPHIC ANALYSIS OF FOUNDATION AND SPILLWAY ROCK

NEW RIVER DAM

(STUDY II)

JANUARY 1985

SAUSALITO, CALIFORNIA

DATE		SPD LABORATORY
GENERAL TEST REPORT	29 JAN 1984	CORPS OF ENGINEERS, US ARMY
	DISTRICT LOS ANGELES	SAUSALITO CALIFORNIA 94966

PROJECT	CONTRACT No.	WORK ORDER No. & DATE
NEW RIVER DAM PETRO ANALYSIS		24 DECEMBER 1984 E86 85 0026

UNIT COST	DATE SAMPLE RECEIVED	LABORATORY No.
$1,500.00	27 DECEMBER 1984	NRD- 7 TO 15

DESCRIPTION	SOURCE
PETROGRAPHIC ANALYSIS OF VOLCANIC ROCK SUITE	NEW RIVER DAM PHOENIX, ARIZONA

PETROGRAPHER
PETER ALMENDINGER/JON ASSELANIS

FINDINGS:

NRD-7 Pyroxene-bearing Andesite

MACROSCOPIC

Medium dark gray (fresh surface) to mottled white to light gray (weathered surface). Aphanitic texture with fine spherulitic mafic minerals appearing as black specks. Sample is nonfractured, very hard and strong, and not weathered. On the fresh cut surface, there is evidence for mineral segregation of light (felsic) minerals into discontinous bands. Hematite occurs in minor amounts throughout the sample.

MICROSCOPIC

Consists of unweathered, anhedral to euhedral phenocrysts of plagioclase feldspar (composition ranges from oligoclase to andesine), and a trace of anhedral to subhedral pigeonite (clinopyroxene, ca-poor augite) and magnetite. The plagioclase feldspar occurs as both twinned and zoned phenocrysts up to 4.5 mm in length. The pigeonite and plagioclase feldspar show no evidence of weathering or breakdown to clays. The matrix consists of subparallel (showing flow direction) microlites of plagioclase feldspar of undeterminable composition with anhedral to euhedral magnetite up to 0.05 mm and anhedral subrounded crystals of pigeonite and rutile. There is no alternation of the groundmass. Approximately 65-70% of the groundmass is composed of plagioclase feldspar with the remainder 30-35% of pigeonite, magnetite, and rutile.

NRD-8 Volcanic Cinder Flow Breccia

MACROSCOPIC

Reddish-brown (fresh and weathered) cinder breccia consisting of andesitic to basaltic scoria blocks and lapilla fragments in a reddish cindery matrix. There is no preferred orientation of fragments or welding of matrix material. Sample strength is weak and has low hardness. Weathering is moderate to deep and the sample is very porus.

MICROSCOPIC

The breccia consists of approximately 60-70% angular to subrounded fragments of highly vesicular andesite or basalt rock with secondary deposits of zeolites (Heulandite or Chabazite) within the vesicles. The basaltic groundmass has been completely devitrified and the microlites of plagioclase altered to albite. The matrix consists of devitrified glass, fragments of andesitic and basaltic lithologies, and fragments of the larger fragments described above. The vesicles in the groundmass have been filled with either Heulandite or Chabazite (CaAl zeolites) while approximately 30-40% of the remaining groundmass has been replaced by muscovite.

NRD-9 Vitric Ash-fall Welded Tuff

MACROSPOPIC

Light purple to purplish-red (fresh and weathered surface) with banding of coarse to fine grain holohyaline glassy ash. The coarser layers have fragments up to 6 mm in length. The presence of dark gray to blackish pumice fragments suggest that the sample has undergone welding anf flattening. Sample is very hard and strong and there is no evidence of weathering. The sample exhibits perlitic texture but no surface flaking. No secondary macroscopic crystal growth is found.

MICROSCOPIC

This sample represents a moderately welded tuff consisting of uncollapsed, collapsed, and welded pumice fragments with subordinate obsidan glass fragments and phenocrysts of plagioclase and alkali feldspar, biotite, magnetite, and minor quartz. The presence of the welded pumice fragments would indicate material from another welded tuff unit. The tuff has been welded into compact glass and subsequently perlitic fractures have developed. Flattening and collapse of pore spaces of the pumice fragment in an approximate length to height ratio of 2 to 6:1 indicates moderate welding. The original pore space of the pumice fragments has been completely eliminated but their structure has been retained. Devitrification of volcanic glass and welded pumice fragments is minimal, only occuring along fragment boundries. The welded tuff has an andesitic composition based on the phenocryst population of 80% plagioclase feldspar, 10% alkali feldspar, and 10% mafics.

NRD-10 Slightly Welded Rhyolitic Ash-fall Tuff

MACROSCOPIC

Light purple (fresh surface) to light pinkish orange (weathered surface) with crude stratification of pumice and glass fragments. The grain size ranges from very fine to medium, with majority of the grain elougated parallel to the crude layering. The presence of dark gray to black elongated layers interbedded with layers of pinkish to purple pumice fragments indicates different degree of welding of the pumice fragments. Stratification is the result of the degree of welding and grain size of the pumice fragments. The sample is moderately to very strong and hard, and shows no evidence of weathering.

MICROSCOPIC

Consists of approximately 50-60% devitrified flattened non-collapse to completely collapsed pumice fragments, 20-30% subangular to rounded fragments of non-devitrified volcanic glass exhibiting perlitic texture, 10-20% phenocrysts of plagioclase and alkali feldspar, quartz, minor mafic and miscellaneous volcanic fragments. Welding is slight to moderate with the flattening of pumice fragment rarely exceeding a length to height ratio of 2:1. Within the phenocryst and volcanic fragment population, approximately 25% are volcanic fragments, 25% quartz, 30% alkali feldspar, and 20% plagioclase feldspar. This would suggest a rhyolitic composition of the phenocryst population. Devitrification is slight to moderate with the formation of devitrification products such as plagioclase feldspar and cristabolite intergrowths absent.

NRD-11 Lapilli-ash Flow Tuff

MACROSCOPIC

Mottled salmon-pink to gray (fresh and weathered surfaces), unsorted and non-stratified. Consists of angular lithic fragments of volcanic origin ranging from 1 mm to 21 mm in size. Sample is moderately hard and strong and shows little to moderate weathering. Surface varnish is yellow-orange in color along with dendritic pyrolusite (Manganese Oxide). Texture indicates slight welding of the ash matrix.

MICROSCOPIC

Consists of coarse ash and lapilli size fragments of undevitrified glass fragments exhibiting perlitic texture, altered and devitrified volcanic rock fragments, uncollapsed and collapsed devitrified pumice fragments, and rounded andesitic rock fragments in an ashy matrix. The ash matrix consists of anhedral to euhedral phenocrysts of 45-50% quartz, 35-40% plagioclase feldspar (varying composition indicates contamination), 15-20% alkali feldspar, and 3-5% magnetite, biotite, or hornblende. Phenocryst percentages indicate a rhyolite to dacite composition of the ash matrix. The majority of the ash matrix is composed of devitrified non-collapsed pumice fragments and non-devitrified volcanic glass. There has been compaction and very slight welding of the particles. A flow origin is suggested by the following evidence: 1) unsorted nature of the deposit and 2) heterogeneous nature of the lapilli fragments and ashy matrix.

NRD-12 Rhyolitic Ash-Fall Tuff

MACROSCOPIC

Light pink to salmon pink (fresh and weathered surfaces), crude to good stratification of well sorted fine to coarse grained ash. The ash consists of dark purple to red glass and andesite fragments and pink devitrified pumice. Sample is well consolidated and moderately hard and strong. Weathering is moderate to deep and can be broken apart under hand pressure.

MICROSCOPIC

Consists of non-flatten, slightly to severely devitrified pumice fragments, non-devitrified subrounded to rounded glass exhibiting perlitic fracture patterns. The sample is composed of approximately 65% pumice fragments ranging from non-deformed, non-devitrified to highly contorted, devitrified with pore spaces completely collapsed, 25% perlitic, non-devitrified glass, 4% miscellaneous rock fragments, and 6% phenocrysts of plagioclase and alkali feldspar, and quartz. Boundries between layers of different grain sizes are sharp and there is no evidence for graded bedding. The individual fragments of glass and pumice are separated by a thin matrix of glass shords and other unidentifiable crystalline mineral. This would suggest that this unit has not undergone compaction and welding as found in other units.

NRD-13 Vesicular Pyroxene-bearing Andesite

Dark brownish-purple (fresh) to salmon-pink (weathered), moderately hard and strong and shows little to moderate weathering. Consists of unweathered microlites of plagioclase feldspar, phenocrysts of hypersthene and pigeonite (orthopyroxene and clinopyroxene respectively), magnetite, and rutile. Randomly scattered phenocryst up to 1 mm of plagioclase feldspar are found. Color difference between NRD-7 and NRD-13 is due possibly to the greater increase in magnetite concentration in NRD-13 and the predominance of plagioclase in NRD-7. Origin is suggested to be primary rather than secondary weathering or reheating by the flow breccia.

NRD-14 Rhyolitic Lapilli-ash Flow Tuff

Mottled gray to pinkish red (fresh and weathered surfaces) in a pinkish matrix. Consists of unsorted, nonstratified, angular volcanic fragments (up to 25 mm) of andesite and basaltic scoria. The sample is friable and is moderately to deeply weathered. Microscopically the sample consists of angular to rounded andesitic fragment (similar to that found in NRD-7 and NRD-13), devitrified vesicular basaltic fragments, and other miscellaneous altered volcanic fragment in a vesicular glassy matrix with a pumice like structure. The matrix had undergone slight devitrification and contains numerous phenocrysts of plagioclase and alkali feldspar, quartz, and biotite.

NRD-15 Andesitic Flow

Banded dark pink to light tan (fresh surface) with occasional microcrystalline phenocrysts of quartz and plagioclase. Sample is very hard and strong, and shows moderate weathering. The flow consists of .5 to 2.0 mm layers of devitrified glassy material with microlites of plagioclase feldspars and phenocrysts of plagioclase and alkalic feldspar, and quartz. Numerous individual and layered vesicle and show development of plagioclase feldspar and cristabolite intergrowths. Evidence for flow origin include 1) continuity of layers, and 2) subparallel microlites of plagioclase feldspar, biotite and plagioclase feldspar phenocrysts.

ATTACHMENT 3

The Earth Technology Corporation

3116 West Thomas Road, Suite 601 • P.O. Box 14570 • Phoenix, Arizona 85063
Telephone: (602) 269-7501 • Telex: 656338

January 23, 1984

U. S. Army Corps of Engineers
New River Dam
P. O. Box 2019
Sun City, Arizona 85372

Attention: Captain Dunn

Subject: Soil Sampling and Laboratory Testing
New River Dam, Arizona
Earth Technology Project No. 84-164-01

Gentlemen:

At your request we have sampled and tested four soil samples obtained from the fracture filling material in andesite bedrock fractures at the base of the core excavation at New River Dam. Samples were obtained December 30, 1983 by our staff geologist, Ron Whitler, under the direct supervision of Corps geologist Bob Thurman. Samples were obtained from fractures ranging from about 1/4 inch to 2 1/2 inches wide. At your request, the following tests were performed on each sample:

- o Atterberg limits
- o Grain size distribution
- o S.C.S. double hydrometer dispersion test
- o Salinity, soluble sulfates, and soluble chloride content

Results of the Atterberg limits, dispersion, salinity, sulfate and chloride content tests are presented in the attached tabulation. Test results were telephoned to the dam site project offices on January 1, 1984. Results of the gradation tests are presented on the attached grain size plot, Figure 1. As requested by the Corps, tests were performed on minus #40 size material to screen out rock fragment contamination picked up during sampling.

Soil descriptions and sample locations are presented below:

Sample No. 1:

CLAY (CH), light greenish gray 5GY 7/1 (Munsell), high plasticity, hard, waxy, noncalcareous, no appreciable coarse sediments except from contamination, trace of hematitic and manganese stains. From bottom of core turn, Station 29+90, 12-20 feet upstream of centerline.

Sample No. 2:

CLAY (CH), mostly light greenish gray 5GY 7/1, but some light gray 5Y 7/1 coloration with 20% of material 7.5 YR 6/8 reddish yellow hematitic coloration, high plasticity, very stiff to hard, noncalcareous, hematite and manganese stains, slightly more porous and less plastic where hematitic colored. From Station 30+20, bottom of core turn, 22 feet downstream of centerline.

Sample No. 3:

CLAY (CH), light greenish gray 5GY 7/1 and reddish yellow 7.5 YR 6/8 with manganese stains, high plasticity, hard, waxy, noncalcareous, slightly more porous than Sample No. 1. From Station 30+70, bottom of core turn, 10-25 feet upstream from centerline.

Sample No. 4:

CLAY (CH), mostly light greenish gray 5 GY 7/1 with 15% reddish yellow 7.5 YR 6/8 and a trace light gray 5Y 7/1, high plasticity, very stiff, noncalcareous, not as stiff as Sample No. 1. From Station 30+90, 12-29 feet downstream of centerline, bottom of core turn.

Please call us if we can be of any further assistance at the New River Dam site.

Sincerely,

Steven A. Haire, P.E.
Project Engineer

SH: jm
cc: R. Roodsari

LABORATORY TEST RESULTS

ANDESITE FRACTURE FILLING AT NEW RIVER DAM

Sample No.	Location	USCS Soil Type	Atterberg Limits LL	Atterberg Limits PI	Salinity (mmhos/cm)	Soluble Sulfates (ppm)	Soluble Chlorides (ppm)	S.C.S. Double Hydrometer Dispersion (%)
1	Sta. 29+90 R 12 to 20'	Clay (CH)	96	64	0.75	127	74	13.7
2	Sta. 30+20 L 22'	Clay (CH)	102	65	0.90	136	82	16.2
3	Sta. 30+70 R 10 to 25'	Clay (CH)	110	74	1.50	165	94	20.5
4	Sta. 30+90 L 12 to 29'	Clay (CH)	115	73	0.65	148	84	18.2

ATTACHMENT 4

REPLY TO
ATTENTION OF

DEPARTMENT OF THE ARMY
SOUTH PACIFIC DIVISION, CORPS OF ENGINEERS
LABORATORY
P O BOX 37, SAUSALITO, CALIFORNIA 94966

SPDED-DL

7 MAR 1984

SUBJECT: New River Dam Arizona

Commander
US Army Engineer District, Los Angeles
ATTN SPLED-GD, A. Roodsari
Post Ofice Box 2711
Los Angeles, CA 90053

1. References:

a. DA Form 2544, CIV 84-53 dated 7 February 1984, requesting testng of soil samples.

b. Samples relative to reference a received on 13 February 1984. Identification of samples is on inclosed plate.

2. Pinhole Erosion Test and Atterberg Limits tests were performed on the above samples in accordance with Engineer Manuarl, EM 1110-2-1906, "Laboratory Soil Testing", 30 November 1970.

3. Soluble salt tests will follow when completed.

4. Total cost of testing is $510.00. Billing will be made by the Sacramento District, Finance and Accounting Branch.

FOR THE COMMNADER:

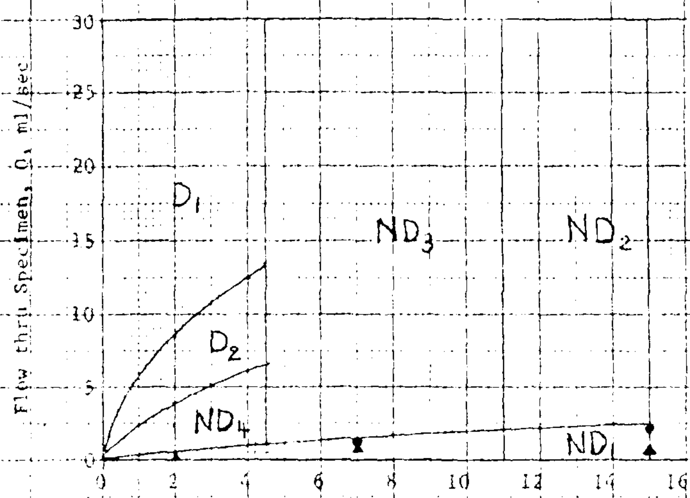
Melvin W Cohen

1 Incl (trip)
as

MELVIN W. COHEN
Director, SPD Laboratory

SOUTH PACIFIC DIVISION
LABORATORY

PINHOLE EROSION TEST



D_1 & D_2-Dispersive and highly erodable
ND_4 & ND_3-Nondispersive intermediate erosion resistant
ND_2 & ND_1-Nondispersive erosion resistant

Div. No.	Hole No.	Depth ft.	LL	PI	Classification	Dry Density pcf	W.C. %	Disp. Class.
86641	20+50	South Trench	112	75	▲ Clay (CH)	83.7	35.8	ND_1
Composite								
86640	30+00	North Trench	109	71	● Clay (CH)	78.0	39.5	ND_1
86642	20+60	"						

Project: NEW RIVER DAM

Date: March 1984

END

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